1922
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WEB STRESSES IN REINFORCED CONCRETE BEAMS

BY

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Ko-Gaku-Shi Imperial University, Japan, 1920

THESIS

SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS
FOR THE DEGREE OF MASTER OF SCIENCE
IN THEORETICAL AND APPLIED MECHANICS
IN THE GRADUATE SCHOOL OF THE UNIVERSITY
OF ILLINOIS, 1922

URBANA, ILLINOIS

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UNIVERSITY OF ILLINOIS

THE GRADUATE SCHOOL

June 3, 1922

I HEREBY RECOMMEND THAT THE THESIS PREPARED UNDER MY SUPERVISION BY SABRO UCHIMURA

ENTITLED WEB STRESSES IN REINFORCED CONCRETE BEAMS

BE ACCEPTED AS FULFILLING THIS PART OF THE REQUIREMENTS FOR THE DEGREE OF MASTER OF SCIENCE IN THEORETICAL AND APPLIED MECHANICS

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*Required for doctor's degree but not for master's

499029

TABLE OF CONTENTS

I. INTRODUCTION

Page No.

1. Preliminary..... 1
2. Scope of Investigation..... 2
3. Acknowledgments 3
4. Analysis or Theory..... 4
   - A. Beams Without Web Reinforcement..... 4
   - B. Action of Vertical Stirrups..... 7
   - C. Beams with Bars Bent Up..... 9
   - D. Action of Bent-Up Bar..... 11
   - E. Tee-Beams..... 12

II. Materials, Test Pieces, Apparatus and Methods of Testing

5. Materials..... 15
6. Test Beams..... 21
7. Making of the Beams..... 22
8. Storage of Specimens..... 26
9. Method of Testing..... 26

III. EXPERIMENTAL DATA AND DISCUSSION

Page No.

10. Description of Figures, Photographs and Tables..... 34

11. Deflection of Beams..... 35

A. Beams Without Web Reinforcement

12. Note of Tests..... 36

a. Beams without Hooks..... 36

b. Beams with Hooks..... 37

13. Beams Without Web Reinforcement..... 39

14. Effect of Hooks at the Ends of Beams..... 40

15. Position of the Neutral Axis..... 41

16. Value of Vertical Shearing Stress..... 43

17. Direction of the Diagonal Cracks..... 46

B. Rectangular Beams with Vertical Stirrups..... 46

18. Note of Tests..... 46

a. Rectangular Beam with 4 in. Spacing of Stirrups..... 46

b. Rectangular Beams with 7 in. Spacing of Stirrups..... 50

c. Rectangular Beams with 11 in. Spacing of Stirrups..... 51

19. Rectangular Beams with Vertical Stirrups..... 52

20. Stress in Stirrups..... 54

21. Effect of Spacing of the Stirrups..... 55

22. Effectiveness of Vertical Stirrups..... 57

Page No.

C. Tee-Beams with Vertical Stirrups.....59

23. Note of Test.....59

a. Tee-Beams with 4-in. Spacing.....59

b. Tee-Beams with 7 in. Spacing.....60

c. Tee-Beams with 11 in. Spacing.....62

24. Tee-Shaped Beam with Vertical Stirrups.....64

25. Stress in Stirrups.....66

26. Position of Neutral Axis.....68

27. Effect of Spacing of Stirrup.....69

D. Rectangular Beam with Bent-up Bars.....72

28. Note of Test.....72

29. Rectangular Beams with Bent-Up Bars.....73

30. Conclusions.....75

LIST OF TABLES

Page

Table 1. Tension of Steel..... 16

Table 2. Tests of Cement and Mortar..... 16

Table 3. Mechanical Analysis of Sand 17

Table 4. Mechanical Analysis of Gravel..... 18

Table 5. Physical Properties of Concrete..... 20

Table 6. Spacing and Size of Vertical Stirrups..... 22

Table 7. Strain Gage Constants 27

Table 8. Value of k and j when n = 7 and p = 0.0233..... 42

Table 9. Calculated Shearing Stresses 46

Table 10.Location of Crack Openings and Their Inclination..... 47

Table 11.Maximum Unit Stress and Total Stress in the Stirrups at the Load of 200 000 lbs..... 55

Table 12.Ratio Between Observed and Calculated Stirrup Stress..... 57

Table 13.Maximum Unit Stress and Total Stress in the Stirrups at the Load of 200 000 lbs..... 67

Table 14.Location of the Neutral Axis in T-Beams..... 69

Table 15.Value of k and j in T-Beams..... 69

Table 16.Maximum Observed and Calculated Stresses in the Stirrups at the Load of 200 000 lb.... 71

LIST OF PHOTOGRAPHS

Page

1. View of Form and Reinforcement for Rectangular Beams.... 23
2. View of Form and Reinforcement for T-Beams..... 24
3. View of Test Beams..... 25
4. General Apparatus for Tests..... 28
5. Preparation of Beams for Testing..... 29
6. Strain Gages..... 31
7. Application of Strain Gage in Testing..... 32
8. View of Beam No.221-1 at Failure..... 79
9. View of Beam No.222-1 at Failure..... 80
10. View of Beam No.223-1 at Failure..... 81
11. View of Beam No.224-1 at Failure..... 82
12. View of Beam No.225-1 at Failure..... 83
13. Beam No. 221-1 W 84
14. Beam No. 221-1 E 84
15. Beam No. 221-1 W 85
16. Beam No. 221-1 E 85
17. Beam No. 222-1 W 86
18. Beam No. 222-1 E 86
19. Beam No. 222-2 W 87
20. Beam No. 222-2 E 87
21. Beam No.223-1 W 88
22. Beam No. 223-1 E 88
23. Beam No. 223-2 W 89
24. Beam No. 223-2 E ,,..... 89
25. Beam No. 224-1 W..... 90
26. Beam No. 224-1 E..... 90

			Page
27.	Beam No. 224-2	W	91
28.	Beam No. 224-2	E	91
29.	Beam No. 225-1	W	92
30.	Beam No. 225-1	E	92
31.	Beam No. 225-2	W	93
32.	Beam No. 225-2	E	93
33.	Beam No. 226-1	W	94
34.	Beam No. 226-1	E	94
35.	Beam No. 226-2	W.....	95
36.	Beam No. 226-2	E	95
37.	Beam No. 227-1	W	96
38.	Beam No. 227-1	E	96
39.	Beam No. 227-2	W	97
40.	Beam No. 227-2	E	97
41.	Beam No. 228-1	W	98
42.	Beam No. 228-1	E	98
43.	Beam No. 228-2	W	99
44.	Beam No. 228-2	E.....	99
45.	Beam No. 229-1	W	100
46.	Beam No. 229-1	E	100
47.	Beam No. 229-2	W	101
48.	Beam No. 229-2	E	101

I. INTRODUCTION

I. INTRODUCTION

1. Preliminary.- In a reinforced concrete beam having a depth relatively large in comparison with its length the resistance to web stresses becomes very important, and diagonal tension and bond stresses may control the strength of the beam. At the present time, there is a considerable amount of information available concerning the resistance of web reinforcement to the strains due to diagonal tension, but conclusive data are still lacking on a number of questions such as the action of stirrups in deep beams and T-beams.

High diagonal tensile stresses are developed when the vertical shear is large in comparison with the bending moment: this condition is found in short, deep beams. Tests by W. A. Slater for the Emergency Fleet Corporation in 1918 showed that where such high web stresses occurred they could be resisted by properly designed web reinforcement, and extremely high shearing unit stresses were developed in the tests, which included beams of both rectangular and I-section. As a result of this investigation the recent proposed specifications of the American Concrete Institute and of the Joint Committee on Concrete and Reinforced Concrete allow a maximum shearing unit stress in reinforced concrete beams of twelve hundredths of the ultimate compressive strength of the concrete, nearly twice as high a value as has been permitted in previous specifications. Tests to determine whether such high web stresses may be developed in the commoner forms of beams are thus of timely interest.

Theoretical analysis of the stress in beams may be applied to some extent to the problem; however, it seems likely that the short, deep beam does not follow the same law of action as the ideal slender one, especially with regard to the distribution of bearing stresses and so-called arch action. The tests described in this thesis were, therefore, made to supplement the theoretical indications with actual observations of the behaviour of full-sized test pieces. The tests form a part of the investigation of web stresses in reinforced concrete beams which has been carried on at the University of Illinois under the direction of Professor A. N. Talbot during the last fifteen years.

2. Scope of Investigation.- The investigation was planned to study the effect of stirrup spacing in deep beams, to compare the web resistance of beams of rectangular section and T-sections, and to determine limiting values of shearing stress which may be developed.

The tests were made on (1) two types of rectangular beams without web reinforcement, (2) three types of rectangular sections and three types of T-sections, with loose vertical stirrups, and (3) one type of beam with reinforcing bars bent up in the outer thirds of the span. Two companion beams were made of each type.

For longitudinal reinforcement, each beam had four high carbon steel corrugated bars. The diameter of the bars was 1 1/4 in. and the sectional area was one square inch, making the percentage of reinforcement 2.33 per cent.

The diameters of stirrups for the various spacings were designed to produce practically the same theoretical stress in the web reinforcement of different beams at a given load.

All beams were made of 1:2:4 concrete. Most of them were tested at the age of from 60 to 62 days. Control cylinders were made with all beams. The beams were tested by applying load in a testing machine and measuring a large number of strains in longitudinal and web steel and in the concrete. These measurements were made by the use of the Berry strain gage. Deflections were also measured and the position and size of cracks was carefully observed throughout the test. The load was increased in regular increments until the beam failed, and a complete set of observations was made after each increment of load was applied.

3. Acknowledgment.- These tests were made as part of the research work of the Engineering Experiment Station of the University of Illinois under the general directions of Professor A. N. Talbot. The specimens were made under the supervision of Professor F. E. Richart of the Engineering Experiment Station. Mr. R. L. Brown, Research Assistant in the same station gave many helpful suggestions and much valuable assistance both in making the tests and in working up the data. Mr. N. B. Green, Research Graduate Assistant in Theoretical and Applied Mechanics assisted in making the tests. To these people the writer expresses appreciation for the assistance rendered.

4. Analysis or Theory.-

A. Beams Without Web Reinforcement.- In a simply supported reinforced concrete beam under loads, there are tensile strains on the convex side and compressive strains on the concave side, and diagonal tensile strains in the web of the beam between support and load point. For use in finding the stress at the top of the beam the assumption that an initial plane section remains plane under flexure may be allowed without any practical objection, and if the stress strain relations found from cylinder tests are used to modify the variation of stress in the beam, a satisfactory analysis of the compressive stresses in concrete may be made.

The tensile strength of concrete is very small, perhaps one-tenth of the compressive strength, so that at ordinary working stresses in beams the concrete has failed at many places in tension. Tests show that as the loads are increased on a beam, the reinforcing steel develops an increasing proportion of the resisting moment, until at the ultimate load it furnishes the entire tensile resistance of the beam. For most purposes, therefore, the assumption of no tensile resistance in the concrete is a satisfactory one, and errs, of course, on the side of safety.

The web stresses in a beam having no web reinforcement may be considered in two stages, (a) before cracks have developed and (b) after development of diagonal tension cracks.

An idea of the nature of the web stresses may be

obtained by comparison with the web stresses in a homogeneous beam. In such a beam if the tensile unit stress at a given section is S and shearing unit stress is t, by the ordinary formulas for combined stresses, there follows

$$S_z = \frac{S}{2} + \sqrt{\frac{S^2}{4} + t^2}$$

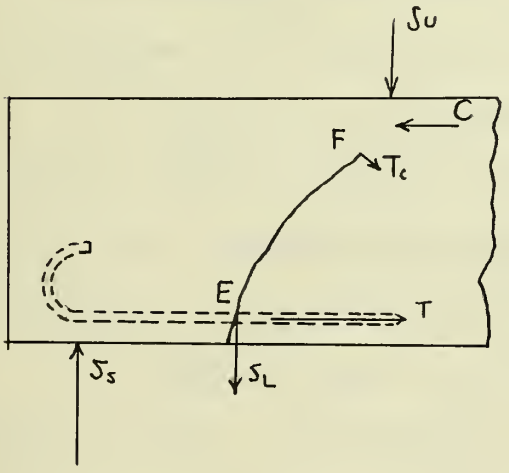
$$t_z = \sqrt{\frac{S^2}{4} + t^2}$$

where S_z is the maximum tensile stress on a section inclined to the direction of the tensile stress and t_z is the maximum shearing stress on another inclined section. The maximum inclined tensile stress is naturally of prime importance; it is found that the angle of inclination is smallest near the tension surface, so that cracks may start nearly vertically at the plane of the reinforcement, then later extend in the diagonal direction. It might be noted that the expression for diagonal tension also holds for diagonal compression if the corresponding compressive stress is inserted; the plane of the inclined stress, however, is at right angles to that of inclined tension.

In the second stage, after a number of small diagonal cracks have appeared it is obvious that no theory of combined stresses can be applied. The vertical shear is carried by the unbroken section at the top of the beam and by the stiffness of the horizontal reinforcing bars acting somewhat as dowels. If the horizontal bars are anchored by hooks, this portion of the shear transferred by the horizontal rods may bequite large.

The analysis of short deep beams is further complicated by the fact that high bearing stresses are developed

at the supports and load points. These high load stresses, although perpendicular to the ordinary compressive and tensile stresses, undoubtedly have an effect upon the resistance of a beam. Furthermore, with the horizontal steel well anchored, it seems likely that in the later stages of loading, truss or arch action is developed to some extent.



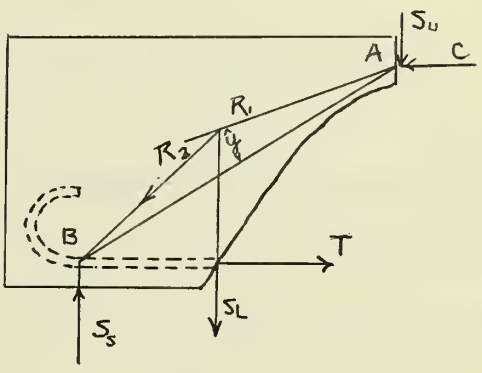


Fig. 1 . Fig. 2

Assume that a large diagonal tension crack appears near the load point as shown in Fig. 1.

Now the portion of the beam to the left of this crack is taken as a free body as shown in figure 2. It is seen to be subject to the following forces: the compression c in the concrete at the top, the tension T in the steel at the bottom and perhaps a very small amount of tension Tc, in the concrete at the end of the crack, a shearing stress S due to load, partially carried by horizontal reinforcement, at the top vertically and reaction S_s at the bottom of the beam acting as external forces. These forces are, as shown in the

Fig. 2 , acting as a so-called arch action and to hold the equilibrium condition. This free body should act as an arch or as a curved column having tension at the top and compression at the crack side, and its resisting moment should be equal to Ry. Where R = Resultant of Su and C (Tc may be negligible) and some times + Sc .

y = deflection of this stress line from AB in Fig.2 If the Sc becomes larger, the y becomes larger and if the crack opens nearer the support, y becomes larger.

As there is no shear on the surface EF and also no tension there, the only force acting to widen the crack opening will be that producing the deformation of steel reinforcement at the bottom, if the effect of S or the arch action is very small.

Then if there is no specially weak part in the steel at that point , new diagonal tension cracks may appear until the Sc becomes large enough to be taken into consideration.

After the arch action become large, the opening of this crack only becomes wider as the load increases, while the outer cracks will probably develop slowly due to relieving action from this crack.

If the shearing strength in the concrete at the top is strong enough to resist shearing failure under higher loads, horizontal tension cracks may be afterward observed between concrete and steel starting from the vertical crack and this may cause the failure of the beam.

B. Action of the Vertical Stirrups.- Until the formation of diagonal cracks the web stresses are carried

mainly by the concrete. But after a crack has formed, the total shearing stress may be carried by the unbroken area of concrete and longitudinal reinforcement and also by the web reinforcement when there is any.

The part of the shearing stress carried by the longitudinal steel contributes the greater part to the cause of diagonal tension failures, as the amount of shear which can be transferred by the steel is limited by the vertical tensile stresses in the concrete in this horizontal section. These shearing stresses are large because they are concentrated in a small sectional area of steel. Any shearing stress in excess of the amount of the vertical tension in the concrete in this horizontal section must tend to bend the longitudinal steel down.

The vertical stirrups may be considered to act in two ways.

(1) To prevent the opening of cracks which cross the stirrups. For this purpose bond stress is essential. Small spacing and large number of stirrups has an advantage for this purpose. However, on the other side, if the large cracks do not open, and there is no relieving action, many cracks may be observed.

(2) To transfer the shearing stress across the crack from the middle portion of the beam to the end portion, the part of the shearing stresses transferred by the horizontal reinforcement is carried by the action of stirrups to the end portion beyond the crack.

For this purpose, strength of vertical steel or stirrups control the resisting strength of the beam.

The shearing stress distribution in a beam with stirrups may be shown in Fig.3.

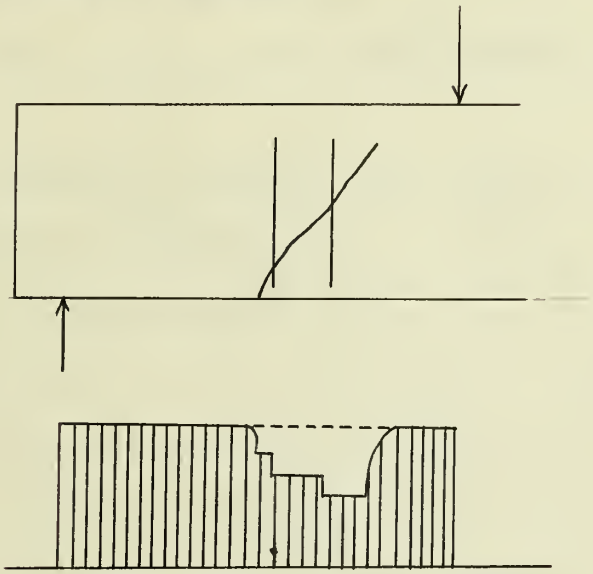


Fig. 3.

C. Beams with bars bent up.- Generally speaking, the distribution of web stresses in beams with bars bent up is very complicated and indeterminate. Due to distortion which takes place in the concrete the assumption that a plane section remain a plane section after bending may not be exactly true, but it is used in the following analysis.

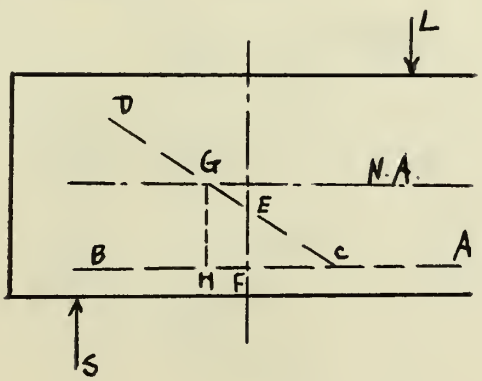


Fig. 4.

In Fig. 4 consider that part of the bars are bent up along CD and that the remainder run straight toward the end B. At A these bars have same amount of tensile stress. However, at the point E the bent up bar carries lower stress than the straight bar carries at F, and the tensile stress caused by bending moment is zero at the point G.

The bent-up bar carries some amount of shearing stress, especially after the crack is developed. However, the bond stress from C to G may be very large, and also the stretching action at C; the bond stresses in this part may be one of the weak points of the beam. In the straight bars at the bottom, the stress at F sometimes may not be much less than at C, since at B it is taking the larger part of the bending moment, and hence the bond stress developed in the portion CG will be less than that found in the case where all bars are straight, while beyond H and toward B the decrease in stress will be rapid and the bond stress developed will be correspondingly greater.

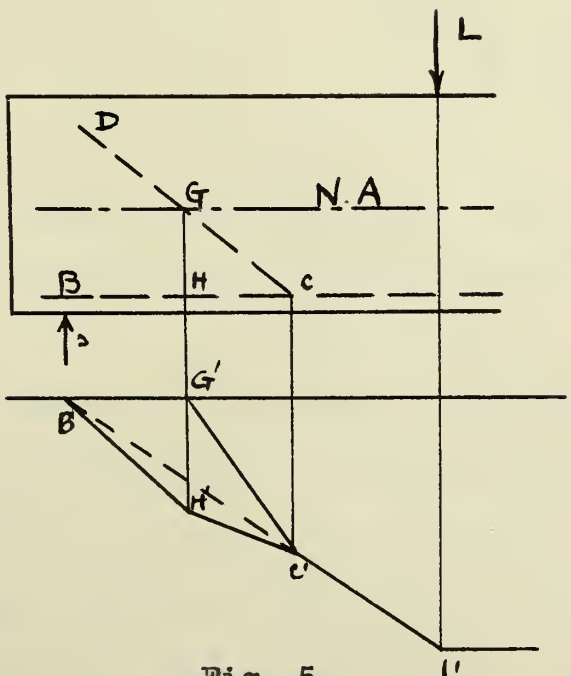


Fig. 5.

Fig. 5 shows the stress diagram in the straight bars and bent up bars.

It would seem, then, that the bending up of the bars results in greater vertical shearing stresses between C and G, and that a large part of the diagonal tension developed here will be caused by the bent reinforcement: and also that in the lower set there is less bond developed between C and H and hence the diagonal tension throughout this portion is less than might otherwise be expected. If, now, another bar be bent up, the stress diagram is shown in Fig. 6.

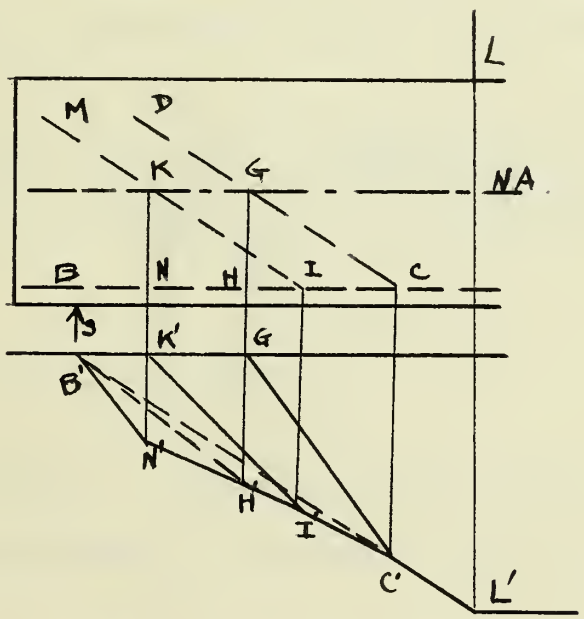


Fig. 6.

D. Action of Bent-up Bar.- The action of a bent-up bar may also be divided in two parts as that of the vertical stirrups.

(1) To prevent crack openings which cross the bent-up bars. For this purpose, if the angle of bend is proper, the bent up bar may act better than the vertical stirrups.

(2)However, for transferring the shearing stresses to the end portion from the other, stirrups have more advantages

than the bent-up bar.

E. T-Beam.- The vertical axis of a T-beam may lie in the flange or below the flange in the web, depending upon the relative depth of flange and beam and amount of steel used. If the neutral axis is in the flange, then the analysis for rectangular beams apply as the concrete below the neutral axis is of no significance.

If it lies in the web, the compression area is different from that in rectangular beams and analysis is somewhat complicated.

Following formulas are obtained by using parabolic formula.

$$k = \frac{\left[-pn + \frac{b'-b}{2b}\left(1-\frac{2}{3}q\right)\frac{t}{d}\right] + \sqrt{\left[\left(pn + \frac{b'-b}{2b}\left(1-\frac{2}{3}q\right)\frac{t}{d}\right]^2 + 2\left(1-\frac{1}{3}q\right)\left(pn - \frac{b'-b}{2b}q\frac{t^2}{d^2}\right)\right.}}{1 - \frac{1}{3}q}$$

$$j = 1 - \frac{1}{3}\left(1 + \frac{1}{4}\,\frac{q}{3-q}\right)k$$

In the T-beam, as the flanges are very large, the shearing stress distribution is something like that shown in Fig. 7.

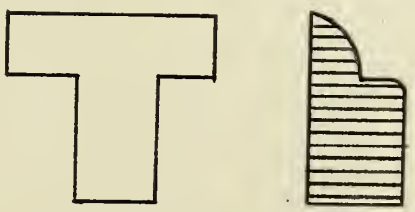


Fig. 7.

The same analysis as applied to the rectangular beam, will be applied to the T-beam, taking the large compressive area at the top and consequently larger value of jd into consideration.

As the shearing stress is very small in the flange shearing failure at the flange may be very rare, and in the deep beam with short span, shearing resistance in concrete is very high. The T -shaped beam may carry a very high load if the web reinforcement used is placed in the proper position.

II. MATERIALS, TEST PIECES, APPARATUS, AND METHODS OF TESTING.

LL. MATERIALS, TEST PIECES, APPARATUS, AND METHODS OF TESTING

5. Materials. - The materials used in making the test beams were similar in character to those used for several years past at the University of Illinois in making reinforced concrete test specimens, and they may be considered to be representative of the best materials used for this class of work in this section of the country.

The cement was furnished by the Universal Portland Cement Company. The mild steel rods used for web reinforcement were furnished by the Illinois Steel Company. The corrugated bars used for longitudinal reinforcement were supplied by the Corrugated Bar Company.

Steel. - For horizontal reinforcement 1 1/4in.round high-carbon corrugated steel bars were used, and for vertical strings 3/8, 1/2, and 5/8 in. round mild steel bars were used. Coupons for tension tests were cut from the ends of bars used in the beams. The results of the test were quite uniform. Table 4 shows the average data of the tests.

TABLE 1

TENSION TESTS OF STEEL

No.	Diameter of bar	Kind of steel	Unit stress at yield point lbs. per sq. in.	Ultimate strength lbs. per sq. in.	Percent elongation in 8 in.	Percent reduction of area
1	3/8	Mild	42,880	57,830	30.2	67.3
2	1/2	"	40,120	57,700	29.2	67.8
3	5/8	"	39,610	61,270	28.3	63.9
4	1 1/8	High Carbon Corr.	52,400	88,570	21.3	26.7

Cement. - Universal portland cement was used. This cement was new, having been received from the factory in December, 1921. It passed satisfactorily the tests required by the standard specifications for cement of the American Society for Testing Materials. The water required for plasticity by the Vicat test was found to be 24.5 percent by weight. The specific gravity by the Le Chatelier flask test was 3.12. Tests of tensile and compressive strength are given in Table 3, wherein the values represent the average of tension tests on six briquettes and compression tests on five 2 x 4-in. cylinders. The mortar mixtures are 1:3 by weight.

Table 2

TESTS OF CEMENT AND MORTAR

Material	Tensile strength lb. per sq. in. 7 days	Tensile strength lb. per sq. in. 28 days	Compressive strength lb. per sq. in. 28 days
Neat Cement	—	697	8345
1:3 Ottawa sand mortar	214	311	3320
1:3 Attica sand mortar	309	488	4220

Fine aggregate. - The sand used was of good quality, clean, hard and well graded, obtained from near Attica, Indiana. It weighed 112 lbs. per cu. ft. compacted in the standard way, and its specific gravity was 2.67. The voids in the dry sand were 33.2 percent. Table 3 gives the results of mechanical analysis of this sand, from which the fineness modulus is found to be 3.35 and the surface modulus, 16.6.

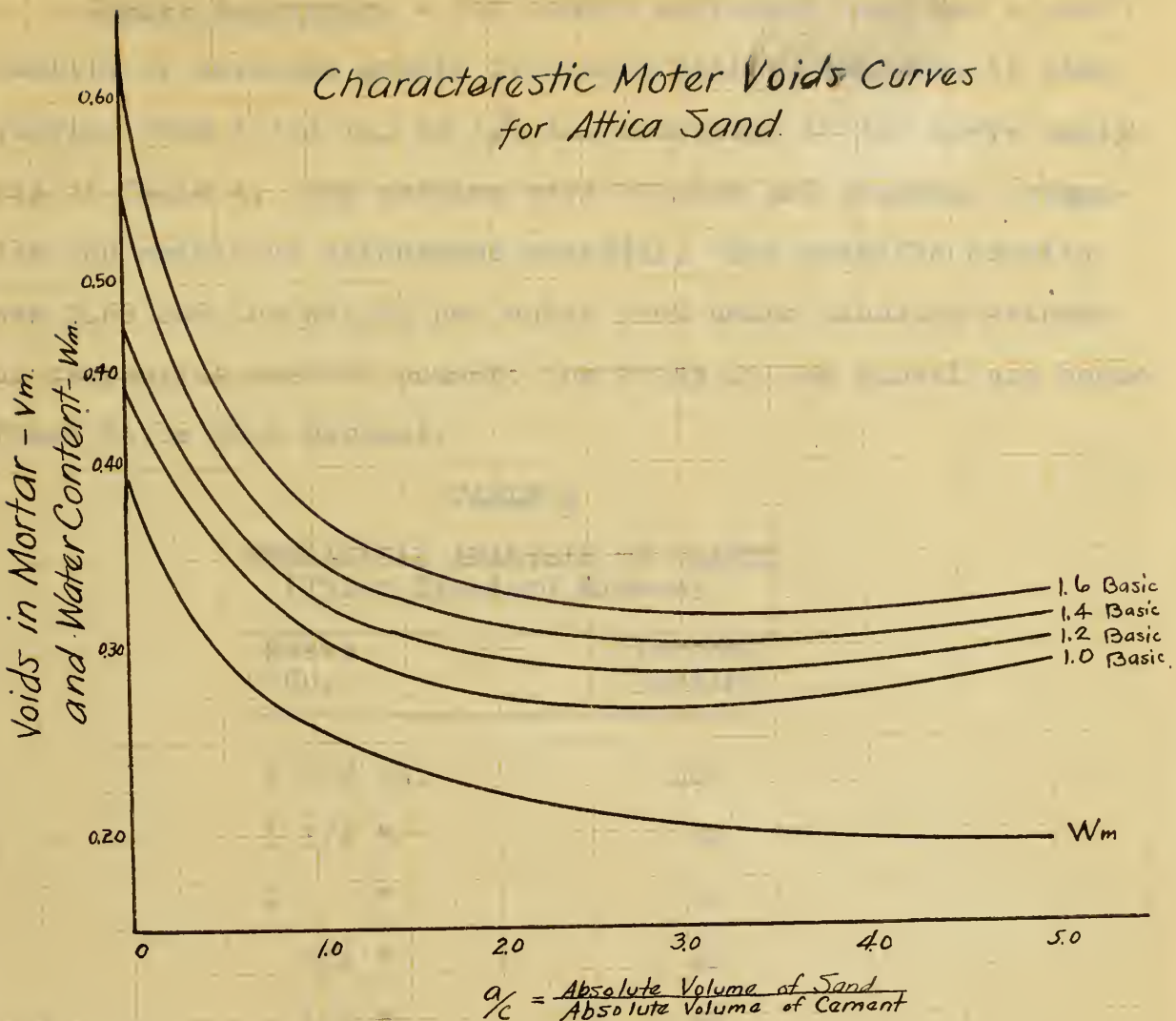
TABLE 3

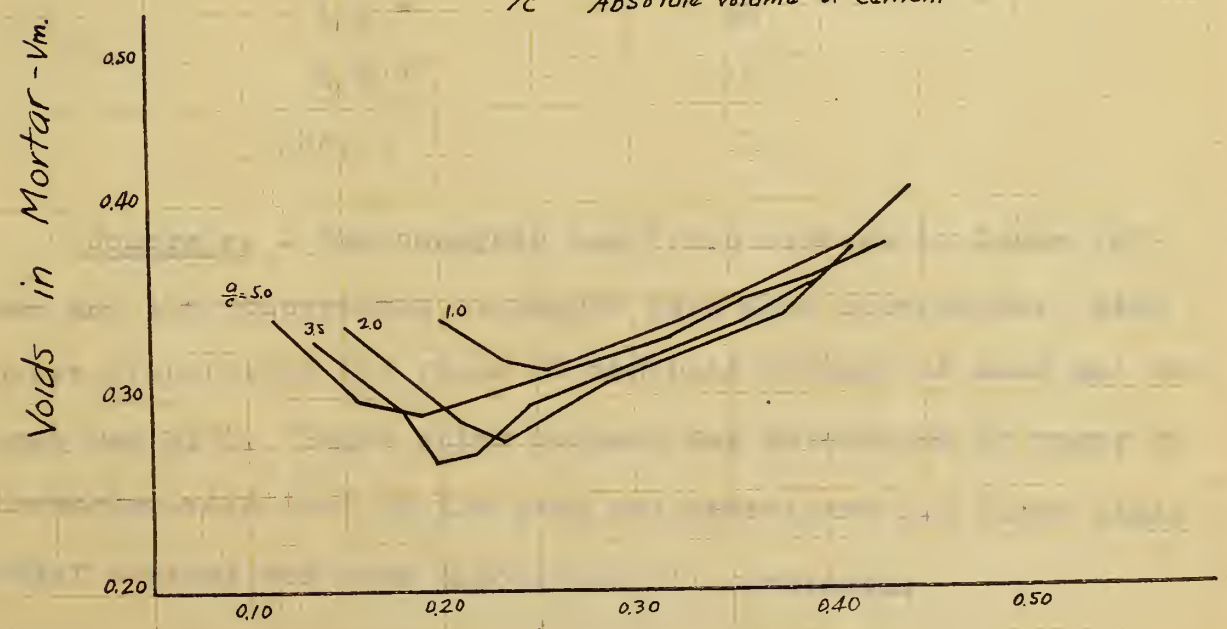
MECHANICAL ANALYSIS OF SAND
(Tyler Standard Sieves)

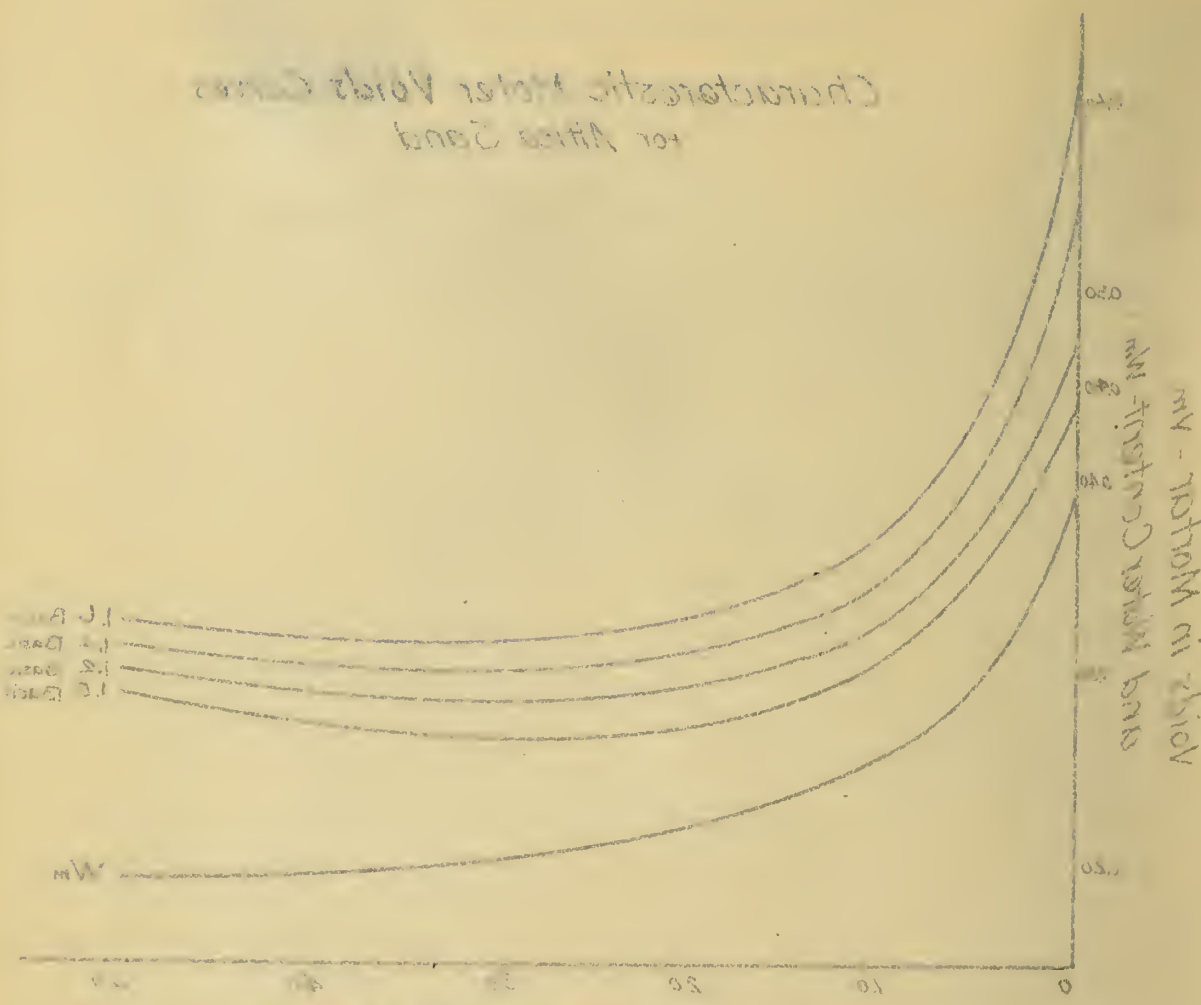
Sieve No.	Percent passing
3/8	100
4	97
8	78
14	51
28	25
48	10
100	4

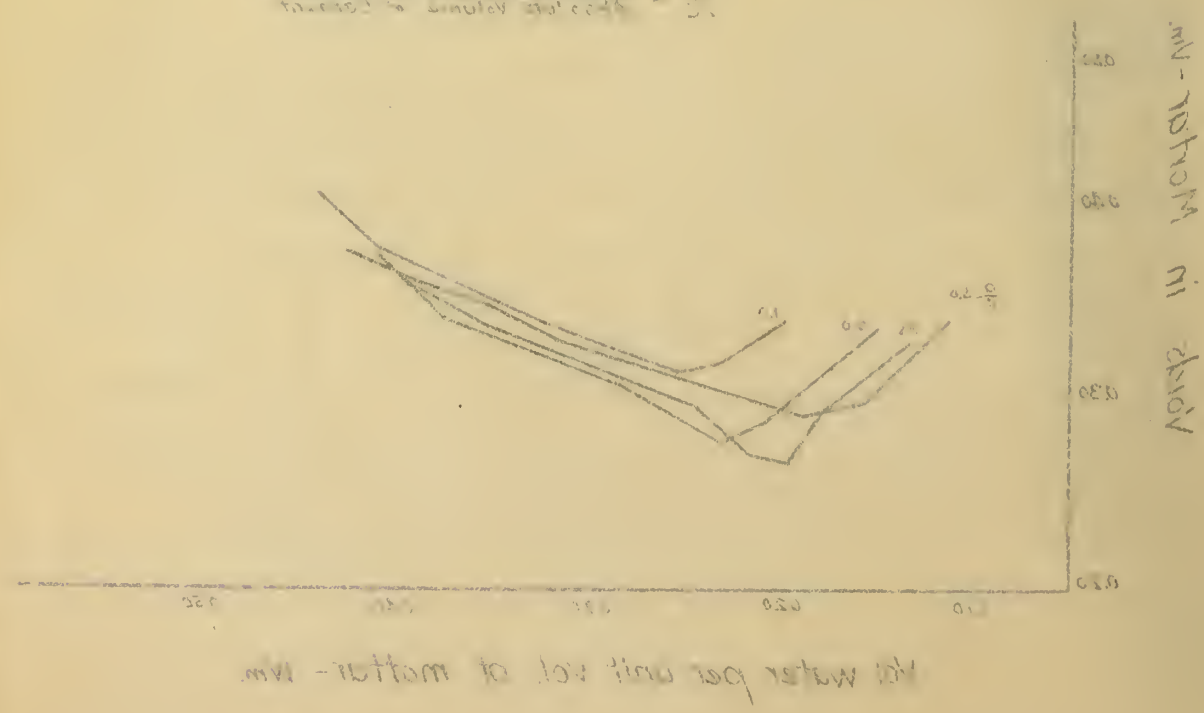
For use in designing the concrete mixture, mortar-voids tests were made using this sand. Curves showing the value of the voids in the mortar, Vm, and the water content of the mortar, Wm, for different mixes and water contents are given on the following page. The volume of water per unit volume of mortar that produces the maximum density is used as a standard for reference and is termed the "Basic" water content.

Characterestic Moter Voids Curves for Attica Sand






Characteristic Water Voids Curves
for Attica Sand
Voids in Mortar - Wm
Wm
0
1.0
2.0
0.20
0.30
0.40
0.50


Voids in Mortar - Wm
Vol water per unit vol of mortar - Wm
0.20
0.30
0.40
0.50

Coarse Aggregate. - The coarse aggregate used was a good quality of screened gravel from near Attica, Indiana, of sizes varying from 1 1/4 in. to 1/4 in., according to the sieve analysis of Table 4. The pebbles were rounded and somewhat irregular and mainly of calcareous material. The specific gravity was 2.69 and the weight per cubic foot under standard methods of compacting was 101 pounds; the voids in the gravel are hence found to be 40.0 percent.

TABLE 4

MECHANICAL ANALYSIS OF GRAVEL
(Tyler Standard Sieves)

Sieve No.	Percent passing
1 1/2 in.	100
1 1/4 "	98
1 "	89
3/4 "	59
1/2 "	35
3/8 "	13
No. 4	3

Concrete. - The concrete was 1:2:4 mixture by loose volume and the proportions by weight were also determined. With these proportions the ratio of absolute volumes of sand and cement was 2.75. Basic water content was determined by means of themortar-voids test on the sand and cement and 1.2 times basic water content was used throughout all specimens.

In making the beams the workability of each batch of concrete was measured by means of the slump and flow table test. In the slump test the settlement or slump of a truncated cone of fresh concrete was measured when the mold was removed. The cone was 12 in. high, and the bases were 4 and 8 in. in diameter. In the flow test a truncated cone about 6 inches high and having bases 6 and 11 in. in diameter was placed on a table, the mold was removed, and the table was raised and dropped thru a distance of 1/2 inch fifteen times. The flow is measured as the ratio of the final diameter to the original diameter of the bottom of the cone. Care was taken in measuring, mixing and tamping to secure as uniform a concrete as possible. Men accustomed to mixing concrete and making test beams were employed in the work. The mixing was done with a Wonder batch mixer, of which the capacity is about 6 cu. ft. The rectangular beams required 4 batches and the T-beams required 5 batches.

For the first batch in each beam, gravel passing a 1 1/2 in. screen was used in order to allow the concrete to flow easily around the reinforcing bars.

In mixing the concrete, the mixer was first washed out, then sand, gravel and cement were introduced in order named, and the mixing water was added last. The batch was then mixed for four minutes, dumped into a tight metal pan, and shoveled into the form. Control cylinders were made with each batch; most of these were tested with the beams but a few were tested at 7 and 28 days. The average strength at 7 days was 1950 lb. per sq. in.; at 28 days, 3230 lb. per sq. in. The average strength

of cylinders tested at the age of the beams with which they were made is given in Table 5 together with the data on slump, flow, and density of the freshly mixed concrete.

TABLE 5

PHYSICAL PROPERTIES OF CONCRETE
(Each value represents ave. of 4 or 5 6 x 12 in. cylinders)

Corres. Beam No.	Age at Test Days	Compressive Strength lb.per sq.in.	Mod.of Elasti-city	Slump in.	Flow Percent of Orig.Diam.	Density of Concrete
221-1	61	4076	4,165,000	0.4	149	——
221-2	61	3696	3,575,000	0.5	164	.882
222-1	60	4522	4,030,000	0.9	151	.899
222-2	59	4337	5,360,000	0.4	155	.866
223-1	60	4124	4,355,000	0.7	156	.890
223-2	61	3689	4,440,000	0.6	158	.886
224-1	62	4106	4,131,000	0.8	149	.877
224-2	61	3790	4,130,000	0.4	157	.880
225-1	62	3788	3,990,000	1.0	147	.892
225-2	61	4041	4,375,000	1.4	159	.882
226-1	62	4037	3,920,000	1.1	152	.889
226-2	60	4331	4,916,000	0.3	145	.879
227-1	62	3799	3,894,000	1.4	158	.880
227-2	60	4346	4,272,000	0.5	159	.882
228-1	62	4058	4,540,000	1.0	150	.887
228-2	60	4152	4,788,000	0.8	168	.882
229-1	62	3931	4,315,000	1.9	163	.882
229-2	60	4203	4,675,000	0.4	153	.882
	Average	4057	4,326,000	0.85	155	.883

6. Test Beams. - Two beams of each of the nine types, a total of eighteen, were made. Three types were T-shaped, and the rest were rectangular beams. The latter have a cross section of 8 x 24 in., and the former were of such a cross section that the flange width was 20 in., flange depth, 6 in., web width, 8 in., and total depth, 24 in. The centroid of the longitudinal reinforcement, which was placed in two layers, was 21 in. below the top surface, and extended the full length of all beams except in No. 229-1 and 2, which had two bent up bars in the outer third of the span length. The percentage of longitudinal steel in all beams was 2.33 percent.

All horizontal reinforcing bars had hooks of 3 1/2 in. radius at both ends except No. 221-1 and 2, which had straight horizontal bars without hooks.

In beams No. 229-1 and 2, having bars bent up, the first bend began at a point 6 in. outside the load point and passed diagonally upward at about 45 degrees with the horizontal axis. A second bar was bent up parallel to the first and so hooked that the highest part of the hook came about 2 inches below the top surface, and 2 in. from the end of the beam. The general arrangement of reinforcement and other details of these beams are shown in Figure 15 In the beams with stirrups, the latter passed under the longitudinal bars and extended to within 1 in. of the top surface. Two U stirrups were used to form the equivalent of a W-shaped stirrup in all cases. The upper ends of all stirrups had hooks of about 1 1/2 in. radius. Table 6 gives the details of the stirrups used.

TABLE 6

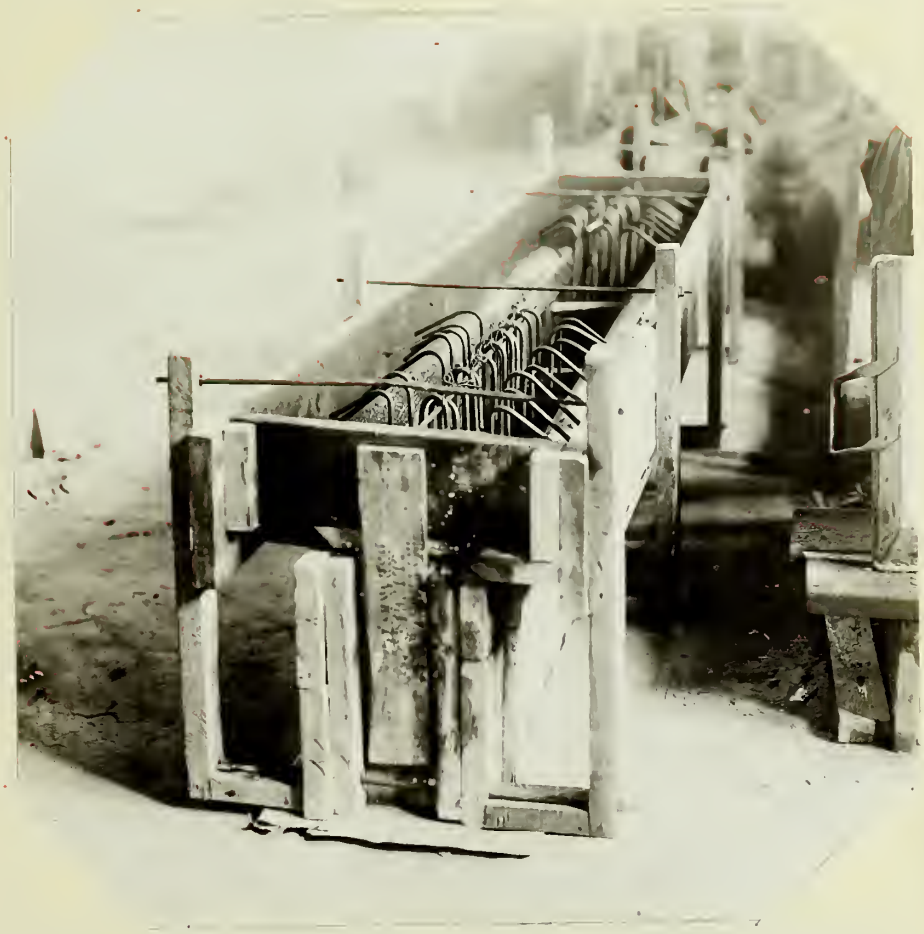
SPACING AND SIZE OF VERTICAL STIRRUPS

Beam No.	Type	Stirrups Spacing-in.	Stirrups Diameter-in.
223-1 & 2	Rectangular	4	3/8
224-1 & 2	"	7	1/2
225-1 & 2	"	11	5/8
226-1 & 2	Tee-Section	4	3/8
227-1 & 2	"	7	1/2
228-1 & 2	"	11	5/8

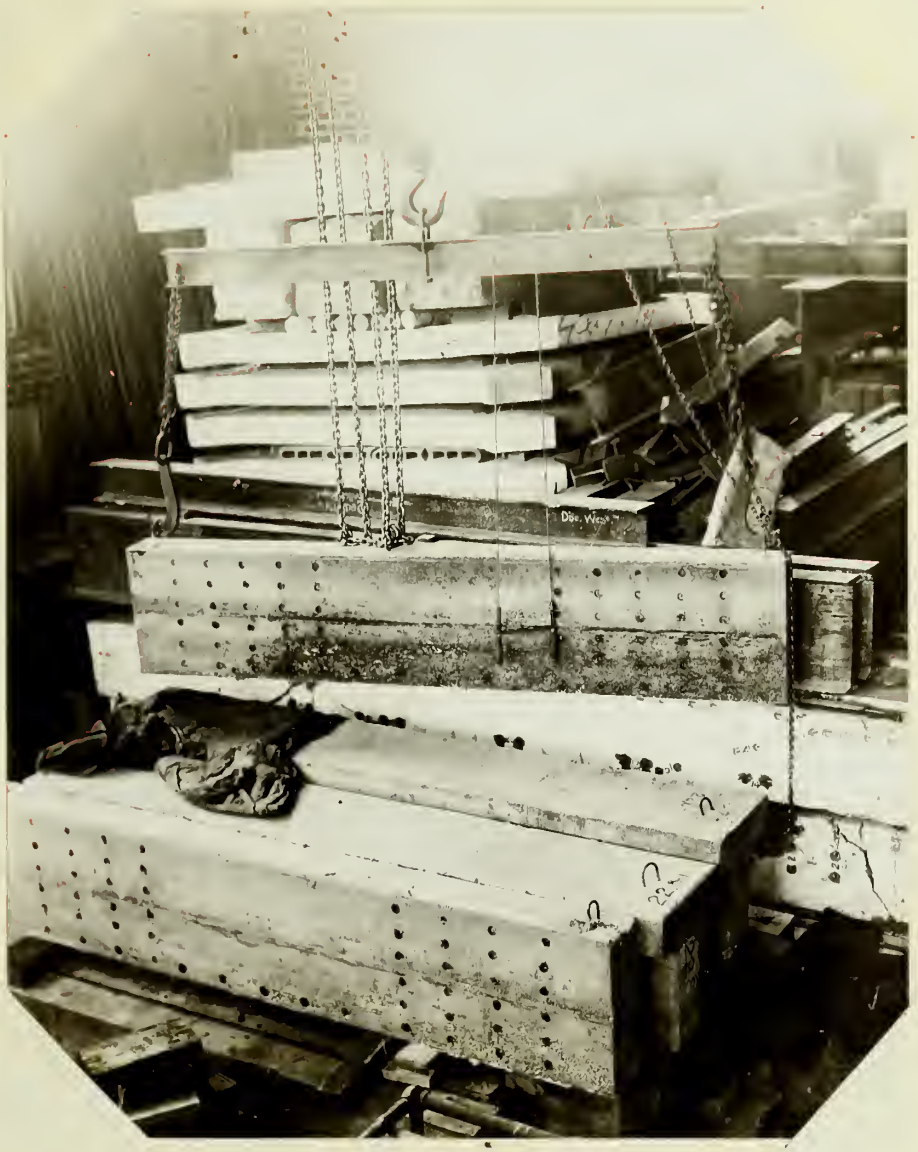
7. Making of the Beams. - The beams were made in built-up wooden forms on the floor of the Concrete Laboratory, being prevented from adhering to the floor by a strip of building paper. The forms are shown by the photographs on following pages. In placing the steel in the forms, care was taken to locate it as nearly as possible in the position shown on the details.

The longitudinal steel was supported by 1/4 in. bars at the proper distance above the bottom of the beam, while the stirrups were wired at the bottom to the longitudinal reinforcement, and at the top to a 1/4 in. horizontal spacing bar. This wiring and bracing kept the stirrups in position during the pouring of the concrete. Corks about 1 1/4 in. in diameter and from 5/8 to 3/8 in. in height for the different diameters of stirrups were tacked to the form in all places where gage points were to be located and served not only to save much chipping in exposing the steel for gage lines, but also helped to keep stirrups in their proper position during pouring.

1. View of form and reinforcement for rectangular beams.



2. View of form and reinforcement for T-beams.



3. View of test beams.

After the steel and corks had been placed, the concrete was poured. In depositing the concrete, care was taken to get it around all of the steel, and it was well puddled to prevent the formation of pockets of any kind. In addition to the tamping, the sides of the form were rapped with mallets to aid in compacting the material and to improve the appearance of the surface.

8. Storage of Specimens. - The forms were removed in five to seven days after the beams were made, and the beams were stored in the laboratory and covered with burlap which was made wet from time to time.

The temperature of the room in which the beams were cast and stored ranged from 68 to 80 degrees Fahrenheit. The cylinders were stored in the damp air of the moist room at a temperature of about 70 degrees F., and were removed to the testing laboratory one day before testing.

9. Method of Testing. - The tests were made in the 300,000 lbs. Olsen testing machine. The span length was 9 ft. The beam was supported at the ends by rocker bearings which in turn rested on the table of the machine, as shown by the photograph on the following page.

The topof the rocker carried a cylindrical steel bearing of 3/4 in. radius, while the bases were slightly curved to allow a rocking action with changes in the length of the lower surface of the beam. The beam was loaded at the one-third points of the span, the load being transferred from the machine through a spherical bearing block, 24-in. I-beam, and two cylindrical rollers. Iron bearing plates 1 x 5 x 8.5 in. were placed between

the beam and the rocker supports. A layer of plaster of paris was placed between these bearing plates, and the beam to overcome uneveness of surface and was allowed to set under such load as came from the weight of the beam and the apparatus used in loading. A load of 5000 lbs. was used as an initial load in order to tighten up all the loading apparatus before starting the test. Deflection was observed at the middle of the span by means of a fine copper wire suspended at constant tension between points over the supports and at the middle of the depth of the beam.

The wire passed in front of a paper scale attached to the side of the beam at midspan. The scale was pasted on the face of a minor and readings were obtained by lining up the thread and its reflection. These readings were accurate to 0.01 in.

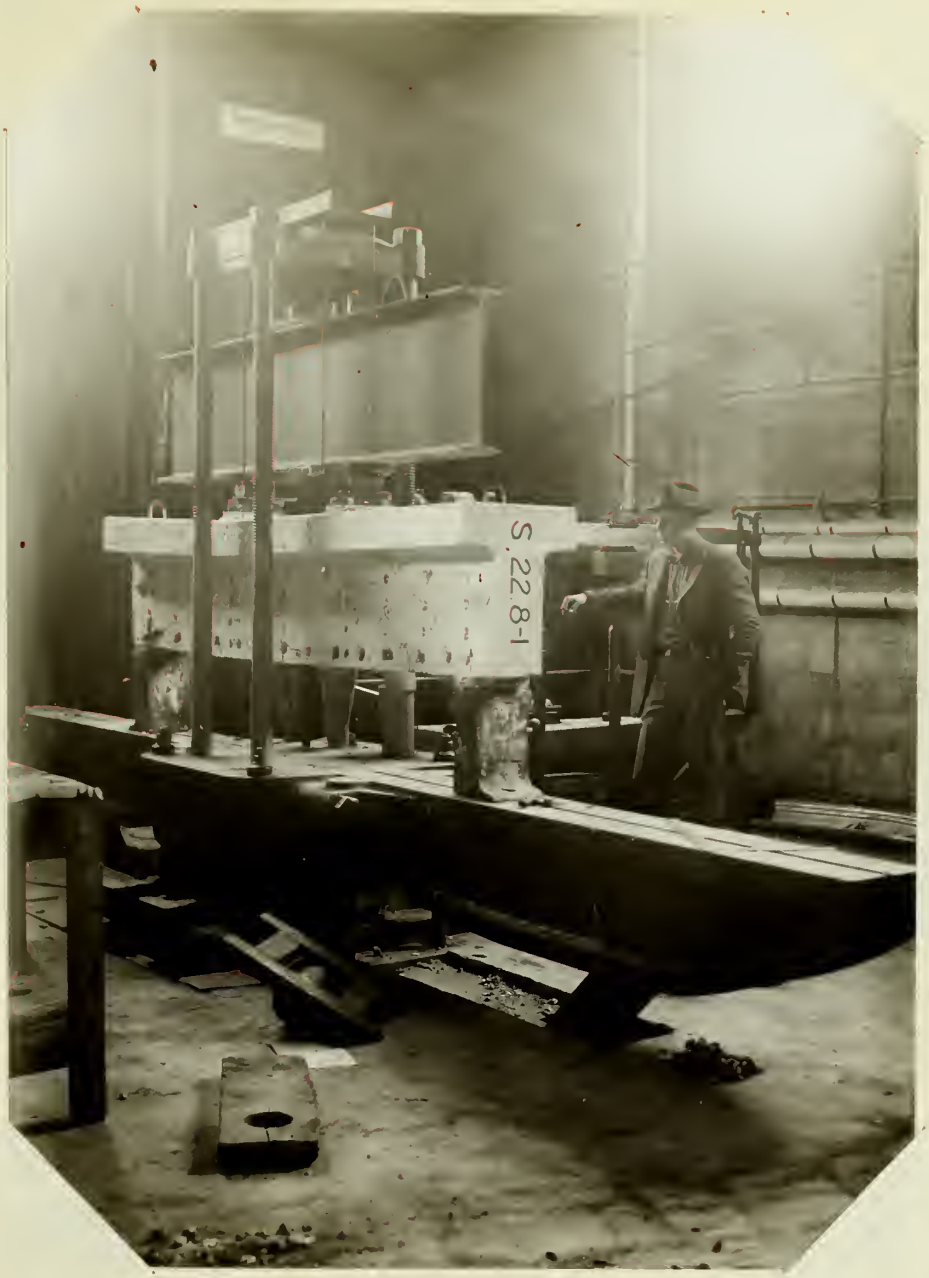
To obtain the longitudinal elongation and shortening at the top and bottom of the beam, the strain gage was used. Four strain gages were used for this test; namely, No. 1, No. 2040, No. 1603, and No. 1604. These are all of the Berry type of strain gage, concerning which information is given in Table 7.

TABLE 7

STRAIN GAGE CONSTANTS

Gage No.	Gage Length	Multiplication Ratio From Calibration	Multiplication Ratio Used for Calculation	Material of Instrument
No. 1	8"	7.587	7.5	Steel
No. 2040	8" and 4"	5.089	5.0	"Invar" steel
No. 1603	4"	7.473	7.5	Aluminum
No. 1604	4"	75.36	7.5	"

The photograph on a following page shows the strain gages and their application.



4. General apparatus for tests

5. Preparation of beams for testing

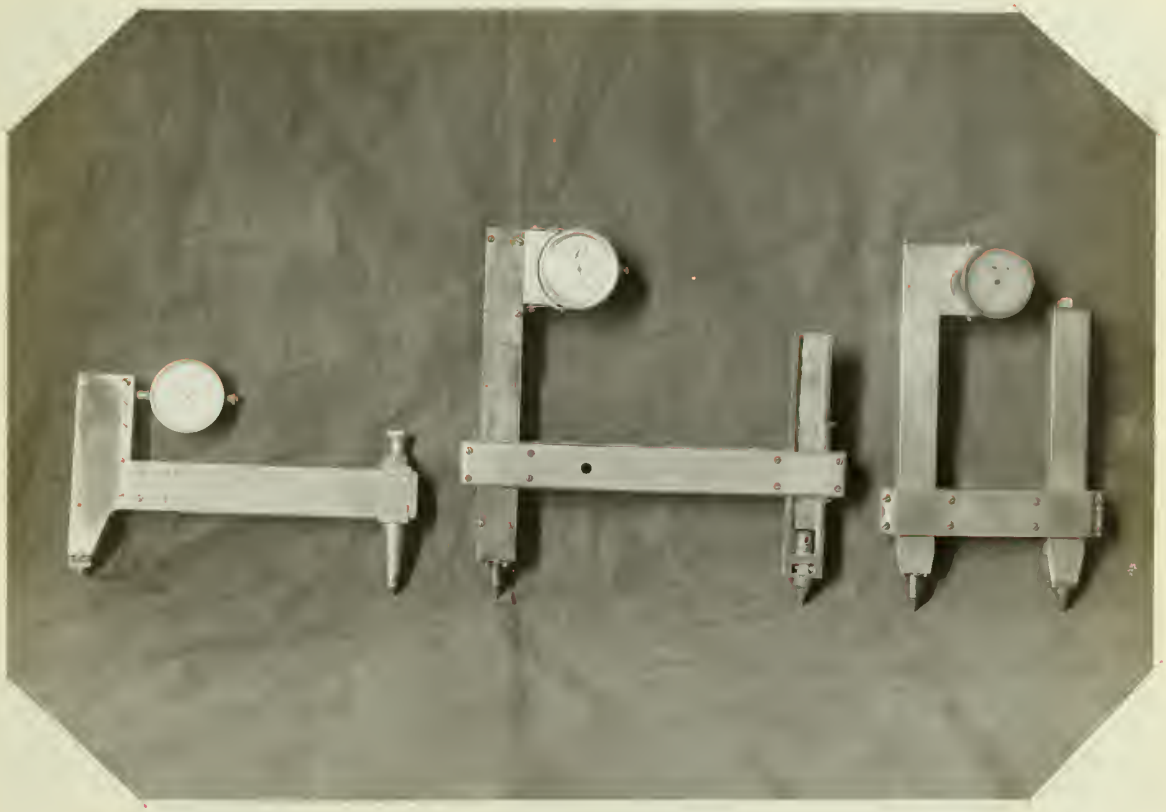
For a more general description of this type of extensometer and its use, reference is made to a paper by Willis A Slater and Herbert F. Moon, in the "American Society for Testing Materials" 1913, on "The Use of the Strain Gage in the Testing of Materials". and to Bulletin No. 64, University of Illinois. The method outlined in these papers was followed in these tests.

Crack openings were measured with a fair degree of accuracy by comparison with the opening of a screw micrometer.

In the test of the first beam, it was found that the most important feature of preparing a beam for strain gage tests lies in the preparation of gage holes.

If the holes were good, the readings could be taken quickly and accurately. However it was impossible to obtain reliable readings from shallow or irregular holes. The gage line is a gaged length between a pair of gage holes and is the length over which the deformation is measured. The gage holes were small holes about 0.05 inch in diameter, drilled in the steel reinforcement and in steel plugs which were imbedded in the concrete at the top surface of the beam.

In order to expose the steel so that gage holes might be drilled, corks were attached to the forms in the proper places before pouring the concrete. After the drilling of the gage holes, the specimens were white-washed before being placed in the testing machine in order that the cracks might be easier to find and mark. Loads were applied at a slow speed of the machine the rate of speed being .05 in. per min. with the machine running idle.



6. Strain gages

7. Application of strain gage in testing

Cracks were noted and marked at loads of 15,000, 25,000, 35,000, 50,000, 60,000, 70,000, 80,000, 90,000, 105,000 and 120,000 lb. and at additional increments of 10,000 lb. Strain measurements were taken at loads of 35,000, 70,000, 105,000, 150,000, and 200,000 lbs. and in the case of higher loads, further readings were taken at the maximum load.

At the initial loading two sets of zero readings were taken for all gage lines and the average of these was used as the zero readings.

In the numbering of gage lines, a uniform system was used. Beginning at the south end on the west side as the beam was set in the machine, gage lines on the first stirrup were numbered from 1 to 5, on the second from 11 to 15, etc., starting from the top. Similarly, starting on the east side at the south end, the gage lines on the first sturrup were numbered from 6 to 10, on the second, from 16 to 20, etc.

On the longitudinal bars all gage lines were numbered from 101 on the west side and from 201 on the east side, numbering from the south end northward; on the compression face the gage lines were numbered beginning with 300 at the south end.

In a few of the tee-beams, additional gage lines on the compression face were numbered beginning with W1 on the west side of the center line, and E1 on the east side; also a few lines set on the sides of the tee-flange were numbered A1 on the west side, and B1 on the east side.

III. EXPERIMENTAL DATA AND DISCUSSION

10. Description of Figures, Photographs and Tables

For use in the explanation of analysis or stress distribution figures have been made and inserted in a convenient place to use.

Typical figures showing all details required for the construction of the beams are inserted on pages 227 to 231.

Seven photographs were taken to show clearly about the equipment used and are inserted in suitable places.

The five photographs shown on pages 78 to 83 were taken to show the typical failures.

Thirty-six photographs on pages 84 to 101 were taken on each side of each beam to show the location of cracks and also the loads as the cracks progressed. The figure on the cracks denote the load in units of 1000 lbs.

Diagram No. 1 shows the characteristic mortar void curves for the sand used in the beams.

Diagram No. 2 shows the observed relation between distance from load point and angle of inclination of cracks.

Diagrams on pages ~~1-8~~ to [illegible] show the relation between the unit stress in the stirrups and the applied load for each gage line.

Diagrams on pages 118 to 226 show the relation between crack opening and stress carried by web reinforcement and horizontal bars.

Beam deflection diagrams are shown on pages 109 to 110

Tables were made and inserted in suitable places when it was thought that they would add to the clearness of the work.

11. Deflections of Beams. - The load deflection diagrams show the deflection of the beams at loads at which the stress measurements were made. It appears that while diagonal tension cracks were developing the rigidity of the outer thirds of the beams decreased somewhat making the deflection of the outer thirds comparatively large as compared to the deflection at mid-span. However, at loads near tensile failure at midspan, the shape of the deflection curve changed again and the deflection at the center increased more rapidly. The difference in the rate of deflection is apparent in the load-deflection diagrams.

The usual calculation of deflection in reinforced concrete beams are based upon the following assumptions:

1. The representative or mean section has a depth equal to the distance from the top of the beam to the center of the steel.

2. The material sustains tension as well as compression, both following the linear law.

3. The proper mean modulus of elasticity of concrete equals the average or secant modulus computed at the working compressive stress.

4. The allowance for steel in computing the moment of inertia of the mean section should be based on the amount of steel in the mid-section.

The effect of cracks is partly covered by neglecting the concrete below the horizontal reinforcement. However, in deep beams with small spans, the effect of diagonal shearing cracks

and deflection due to direct shearing stress may need to be taken into consideration in computing the total deflection.

In this thesis, the limited time does not allow further study of the relations between the deflections and the other elastic properties of the beams.

A. Beams Without Web Reinforcement

12. Notes of Test. - The location of cracks are all referred to the load points and supports. The abbreviation for north-west load point is N.W.L., and of south-east support is S.E.S., etc.

Most of the cracks apparently extended through the thickness of the beam, but in these notes the cracks on each side were treated as if they were separate ones.

a. Beams without Hooks.

Beam No. 221-1: Two cracks appeared at the load of 25,000 lbs. 10 in. from S.W.L. and in mid-span in W. side. At the load of 50,000 lbs. several cracks appeared in both West and East sides, some of them reading about 11 in. from the bottom. The first diagonal crack appeared at North side 10" North from N.W.L. at the load of 55,000 lbs., starting from near the horizontal bar and making an angle of nearly 45 degrees with the horizontal until the middle depth of web was reached; with increasing loads this crack developed both upward and downward.

At the load of 75,000 lbs. diagonal tension crack appeared at both sides at both ends and the diagonal tension crack in N. W. side opened so wide that if the horizontal bars had no corrugations the beam might have failed at a lower load. At the load of 90,000 lbs. cracks reached about 4" below the load point

and shear failure approached. At the load 12,000 lbs. diagonal tension crack opening observed as follows: N.W. .05"; N.E. .04"; S.W. .02"; S.E. .02". The failure came suddenly at the load 154,100 lbs. by slipping of bar combined with shearing failure atthe top of the beam near the load point.

Beam No. 221-2: Cracks did not appear until at the load 50,000 lbs. where a crack under the S.W.L. went up about 12 in. A diagonal tension crack was found at the load 60,000 lbs, went up about 14" high starting from 14 in. South from the S.E.S. toward the S.E.L. At the load 70,000 lbs. a diagonal tension crack appeared in S. W. side and developed quickly. At the load 13,000 lbs. and 14,000 lbs. horizontal cracks at the ends of the beam from 10"to 8" high from the bottom appeared, showing some tensile stress in this part. At 150,000 lbs. when the testing machine was stopped to start measuring strains at this load, the beam failed suddenly by the vertical tension failure at the North end together with shear failure at the top. The vertical tension failure followed very suddenly from this point to the other end of the beam.

b. Beams with Hooks.

Beam No. 222-1: On each side of the beam two large holes under the reinforcement were visible. The first crack appeared at 25,000 lbs. at 6" from S.E.L. under the horizontal reinforcement, and at 30,000 lbs. two cracks were found; at 6" and 14" north from the S.E.L. and 6" south from S.W.L.At the load 45,000 lbs. many small cracks appeared all over the middle third and some of them were found a few inches outside of it. At the load

65,000 lbs. first diagonal tension crack was observed at 14" north from S.W.L. and at the load 70,000 lbs. this crack lengthendd about 24" to a point about 8" under the top surface. At the load 80,000 lbs. a big diagonal tension crack was observed at the northeast side and developed rapidly. The diagonal tension crack in the south side reached a point 5" under the S.E.L. Crack openings were almost same width on both sides; at the load 105,000 lbs. they were about .025" and at the load 135,000 lbs. .05", and at the load 151,000 lbs., .07". Shearing stress in the concrete at the top of the beam near the N.L. caused the failure. It should be noticed that horizontal cracks appeared on the N.E. side at the load of 110,000 lbs. and also after failure tension crack appeared in the top of the beam 9" north from the N.W.L. at the top of the beam. Ultimate load was 167,500 lbs.

Beam No. 222-2: No cracks appeared at the load 25,000, 35,000, 40,000 lbs. First cracks appeared at the load 50,000 lbs. 6" north from S.W.L., 6" high; 14" north from S.W.L., 10.5" high; 14" south from N.W.L., 1" high; 10" north from N.W.L., 1" high; 2" south from S.E.L.,10" high; 1" south from center in E. side, 10" high; 2" south from N.E.L., 10" high; 6" south from N.E.L., 6" high. At the load 70,000 lbs. first diagonal cracks were observed at both ends except N.W. side. At the load 76,000 lbs. big diagonal tension cracks suddenly appeared at the south end and beam of the testing machine dropped for a while. At the load 120,000 lbs. this crack opened 0.075" on the S.E. side. Failure occured at the load 128,000 lbs., caused by vertical shear at the south side load point, load dropped off to 125,000 lbs.

13. Beams Without Web Reinforcement

The dangerous diagonal tension cracks usually appeared about at the middle point of the outer third of the span starting from the reinforcement at the load from 55,000 to 70,000 lbs. It should be borne in mind that the horizontal reinforcement was not plain rods but high carbon corrugated steel bars. On the appearance of these cracks, their direction curved bending toward the load point. As the load on the machine was increased, the cracks lengthened and extended upward to within a few inches under the load point, and the upward extension of this crack almost stopped, while new, large diagonal tension cracks appeared nearer the support, suddenly connecting these former cracks in nearly a straight line, and in some cases the load dropped for a while.

After these cracks were connected, the diagonal tension cracks opened widely, causing the bond resistance between concrete and steel reinforcement to fail suddenly.

If the plain rods had been used instead of corrugated bar as in the longitudinal reinforcement failure might have come at lower loads. Maximum crack opening for these final diagonal tension cracks were observed to be about 0.1 in. wide.

Shearing stress carried by the horizontal bars near the cracks, acted as a tearing stress or a vertical tensile stress of which the intensity is a maximum at the crack and decreased toward the end, between concrete and reinforcement, and whenever tensile strength in the concrete between horizontal reinforcing bars and also between horizontal reinforcing bars and outer surface of the web of the beam, reaches the ultimate strength, the beam may fail by slipping of the horizontal reinforcement.

On the other hand concrete at the top of the beam carrying shearing stress, lost much area as the diagonal tension crack extended toward the load point. If the shearing resistance in concrete reached the ultimate, failure of the beam may be caused by this shear failure.

Failure of the beams with straight bars came suddenly with shear failure at the top and the failure caused by slipping of bars at the bottom at the same time in both cases; it is impossible to find which failure occurred first, and thus to distinguishthe critical source of the failure.

In the beams which have hooks on the horizontal bars, slipping of the bars may be prevented by the hooks to some extent, and failure of this kind of beams are caused by shearing stress in the concrete at the top followed by large cracks appearing at the ends around the hooks.

The beams without web reinforcement carried an average load of 157,000. The average unit shear was 525 lbs. per sq. in. This means 0.121 of the average ultimate compressive strength of the concrete.

14. Effect of Hooks at the Ends of Beams.

Effect of hooks on the horizontal bars prevent slipping of reinforcement in the concrete, consequently crack opening may be prevented to some extent. It may be the reason that the final large cracks started from the points nearer the support in the beams with hooked bars than the one in the beams with straight bars.

However, they did not show any effect materially on the shearing resistance in the concrete at the top of the beams.

Beams with hooks at the ends of the reinforcement all failed by shearing failures at the top of the beam, and did not fail by vertical tensile failure at the bottom of the beam. Therefore in the deep beams for a short span with web reinforcement, it may also be necessary to have hooks at the ends of the reinforcement even when corrugated bars are used.

15. Position of the Neutral Axis

In the beam under flexure, the location of the neutral axis depends upon the ~~strength~~ quality of concrete and reinforcing steel, or in other words, or the ratio of modulus of elasticities between concrete and steel, and also on the amount of steel reinforcement used.

Table 8 shows the values of k and also j. In this table, the tensile resistance of the concrete is neglected. However, in the section where the concrete has some tensile resistance the value of k becomes somewhat larger. The formation of tension cracks may change the position and the curvature of the neutral axis. After a large diagonal tension has developed and arch actionhas developed to some extentin the web of the beam between the load point and the support, the location of the neutral axis may change greatly. Sometimes it may be possible that the neutral axis will cut the top surface in the outer third of the length of the beam though of course this is not the neutral axis of beam action, but it may be called the axis of zero stress.

TABLE 8

Value of k and j when n = 7 and p = .0233

$$k = \sqrt{\frac{2pn}{1-\frac{1}{3}q} + \frac{p^2n^2}{(1-\frac{1}{3}q)^2}} - \frac{pn}{1-\frac{1}{3}q}$$

$$j = 1 - \frac{1}{3}\left(1 + \frac{1}{4}\,\frac{q}{3-q}\right)k$$

q	k	j
0.1	.437	.853
0.2	.442	.850
0.3	.447	.847
0.4	.453	.843
0.5	.460	.839
0.6	.467	.835
0.7	.473	.830
0.8	.480	.825
0.9	.488	.820
1.0	.495	.815

16. Value of Vertical Shearing Stress.- In a beam under flexure, the total tension in the reinforcing bars varies along the length of the beam, as does also the total compressive stress.

The horizontal shearing stress is seen to be necessary in order that the increments or increase of the total tensile stress in the reinforcing bars may be transmitted to the corresponding increments of compression in the compression area of the concrete, the concrete thus acting as the stiffening web of the beam.

The vertical shearing stress is also seen to be necessary in order that the increment of the total moment in any vertical sections between load point and support may be transmitted to the corresponding increments of the horizontal stresses to keep equilibrium condition. For this reason, at any point in a beam, the vertical shearing unit stress is equal to the horizontal shearing unit stress there developed.

If V_1 denotes the part of the total vertical shear which is proportional to the resisting moment attributed to the tensile stresses in the concrete, then

$$v_1 = \frac{V_1}{bd_1}$$

where v_1 = vertical or horizontal shearing stress per unit of area in the concrete due to V_1

d_1 = distance from the center of gravity of compressive stresses to the center of gravity of tensile stresses in concrete.

And also V_2 denotes the rest of the total shear, the amount

being proportional to the part of the resisting moment that is due to the tensile stress in the reinforcing bars.

$$v_2 = \frac{V_2}{bd'}$$

where v_2 = vertical or horizontal shearing stress per unit of area in the concrete due to V_2

d' = distance from the center of the reinforcement to center of gravity of compressive stresses.

If the tensile stress in the concrete is neglected, then

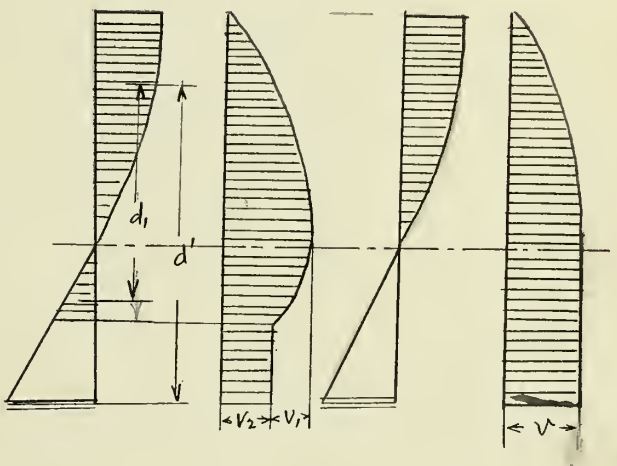
$$V_2 = V$$

and

$$v = \frac{V}{bd'}$$

The distribution of these shearing stresses is shown in Fig. 8

The calculated shearing stresses for these two cases are shown in Table 9 .



(a) (b)

Fig. 8

The following calculations wewre used preparing Table 9. The bending moment,

$$\frac{PL}{2}\times\frac{1}{3}=f_s\, p\, j\, b\, d^2$$

or $$P=f_s\frac{6\,p\,j\,b\,d^2}{L}$$

In the beams tested, p=.0233 b=8in. d=21in. L=108in.

so that $$P=6\times\frac{.0233\times8\times21^2}{108}\,j\,f_s$$

I. Assume $v_1=v_2$ and $2k<1$

$$\frac{P}{2b}=kd\left(\frac{2}{3}+\frac{39}{64}+\frac{1}{8}\right)2v_1+d(1-2k)v_1$$

for j= 0.83 kd= 10 in.

$v_1 + v_2 = .00398P$

for j= .85 kd= 9.3 in.

$v_1 + v_2 = .0044P$

II. Assume $v_1= 0$ $v_2 = v_2$.

for j = .83

v = .00289P

for j = .85

v = .00279P

In Table 9, the second, third, sixth, and seventh columns have been computed on assumption of Case (I), namely $v_1=v_2$. The forth and eighth columns have been calculated on the assumption that $v_1=0$, as in Case (II) above.

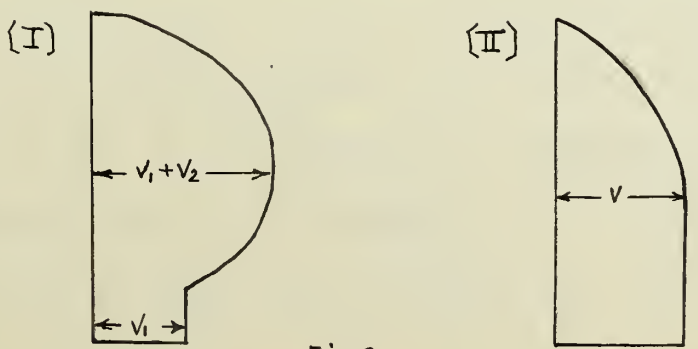


Fig 9.

Table 9.

Calculated Shearing Stresses.

P = 2V	j = .83				j = .85			
	v_1+v_2	v_2	v	fs	v_1+v_2	v_2	v	fs
35 000	139	70	101	9130	154	77	[illegible].5	9350
50 000	199	100	144	13040	220	110	139.5	13350
70 000	279	140	202	18260	308	134	195.2	18700
100 000	398	199	289	25980	440	220	279.0	26600
125 000	498	249	361	31750	550	275	349.0	33250
150 000	597	299	433	38950	660	330	419.6	39900

17. Direction of the Diagonal Cracks.- Cracks appearing in the outer third have certain inclinations from the vertical direction due to maximum stress line and the farther their location from the load point, the larger the inclination from the vertical direction. Table 10 and Fig. [illegible] show the relation between inclination and distance from the load point of the cracks in the beams tested. Though these data were not enough to drive conclusive opinion they are given out in the hope of throwing some further light on this problem.

B. Rectangular Beams with Vertical Stirrups.

18. Notes of Test.- Rectangular beams with 4" spacing of the stirrups. Beam 22.3-1. First two small cracks appeared at 6 in. and 14 in. south from N.E.L. at the load of 25 000 lb. At the load 935 000 lb. there were some strange phenomena; comparatively large tension cracks in the middle third and very small diagonal tension cracks qppeared near the load section. At the load of 70 000 lb. the diagonal cracks became plainly visible and could be traced about 14 in. length. As it seemed impossible to finish the test of this beam that day, the load was released to

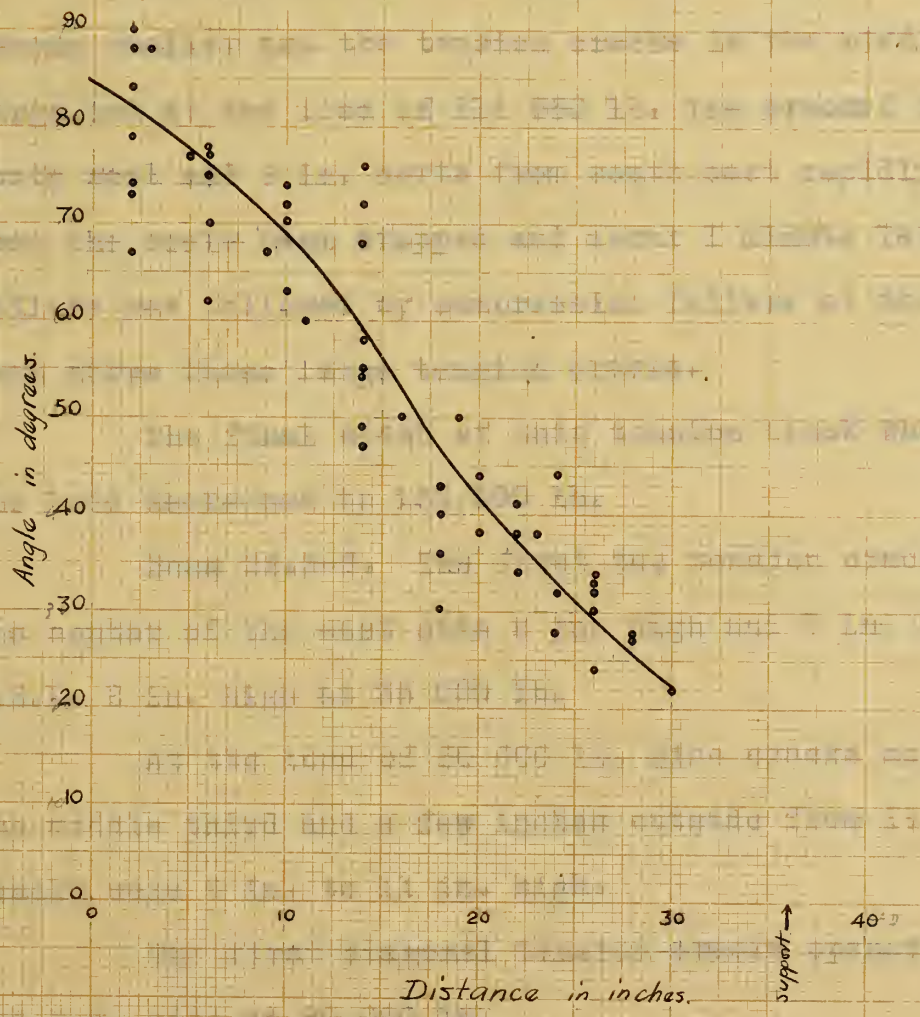
TABLE 10.

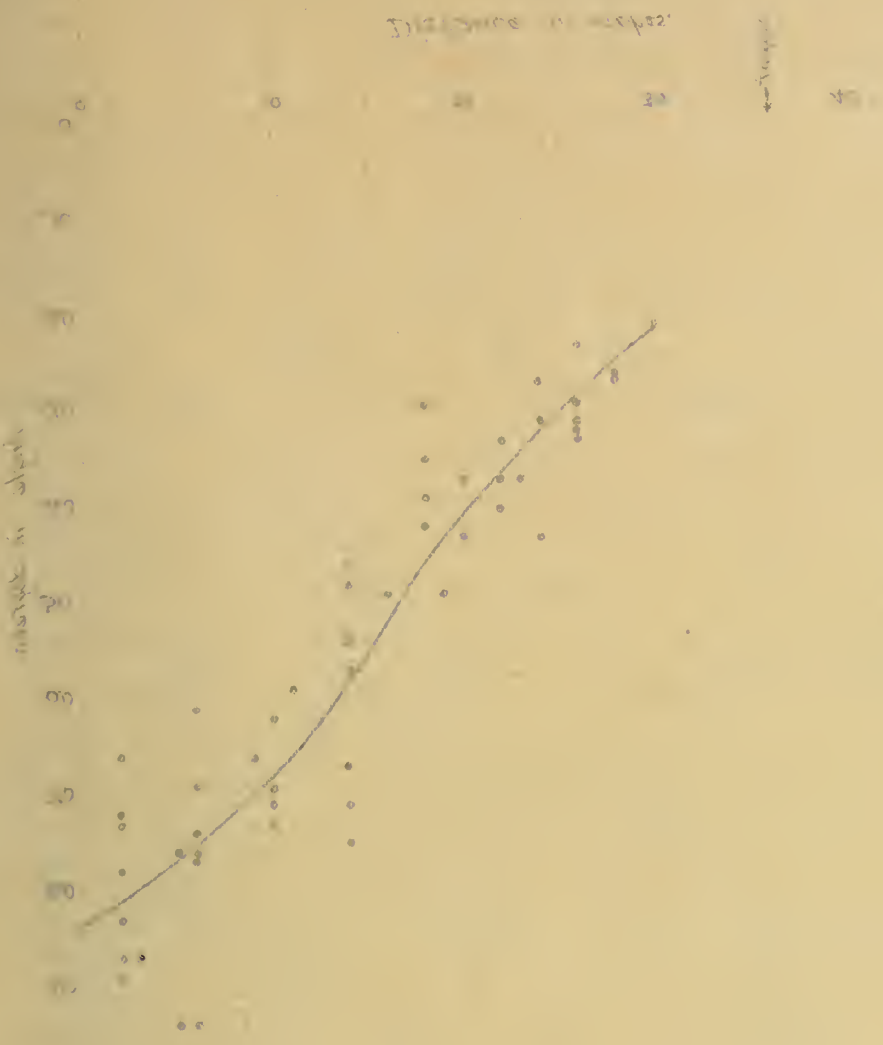
Location of crack opening and their Inclination

Distance from Load Point in inch.	221-1				221-2				222-1				222-2			
	E		W		E		W		E		W		E		W	
	N	S	N	S	N	S	N	S	N	S	N	S	N	S	N	S
1				*	*											
2				67	84		73					88		79	90	74	82
3									88								
4			77								95						
6	70	95								75		78	62				76
8															61		
10			72	63			70		74		70						69
12							60										
14	76	68			58				55	72	54		49	47			59
16						50											53
18				40			43						30		36		
20							38			44							42
22	41		34	38		34			43								37
24					32					28	38			44			33
26	33	34		32								38				24	29
28						28			27								26
30				22			22				22						22
32																	
34																	
36																	

* Note; angles of inclination with the horizontal in degrees.

Relation Between Inclination Of Cracks And Their Distances From Load Point





16 300 lb. and left on until Monday, two days after.

The readings were checked on Monday at the load 70 000 lb. and even the reading in horizontal reinforcement could be checked closely; the maximum difference was 2000 lb. per sq. in. in the center of the span.

At the load of 105 000 lb. the maximum width of the crack opening was 0.01 in. at the S.E. However, beyond this load the cracks did not open any wider.

At 210 000 lb. the diagonal tension cracks seemed to become smaller and the tension cracks in the middle third became large and at the load of 214 550 lb. the cracks 5 in. north from south west and 6 in. north from south east rapidly opened wider. Then the scale beam dropped and about 1 minute later this tension failure was followed by compression failure at the top of the beam above these large tension cracks.

The final width of this tension crack was about 0.085 in. The load decreased to 180,000 lb.

Beam 22.3-2. The first two tension cracks appeared at the center of the east side 2 in. high and 2 in. south from the N.E.L. 2 in. high at 35 000 lb.

At the load of 50 000 lb. nine cracks appeared between the middle third and a few inches outside from it. Most of these cracks were 9 in. to 11 in. high.

The first diagonal tension cracks opened in S.E. side and N.W. side at 80 000 lb.

At the load of 210 000 lb. the maximum tension crack was widened to about 0.01 in. at S.W. side, and at the load of 218 400 lb. tensilestress in the horizontal reinforcement caused

failure at first and was followed by crushing of the concrete at the top of the beam above this crack. The load decreased to 120 000 lb.

b. Rectangular beams with 7 in. spacing of the stirrups. Beam 224-1. This beam had not a good surface on the web and it was very hard to distinguish the cracks.

A very small crack appeared 2 in. north from S.E.L. about 1 in. high at 35 000 lb. At the load of 45 000 lb. two cracks appeared 6 in. from S.E.L. and S.W.L. about 2 in. high. At the load of 55 000 lb. these cracks lengthened and a new crack appeared in the middle third.

The first diagonal cracks appeared at the south side at the load of 80 000 lb. After this load, the vertical cracks did not lengthen so much and diagonal cracks appeared rapidly.

At the load of 200 000 lb. the maximum width of the diagonal tension cracks became .005 in. However, after this load the diagonal tension cracks rather closed to some extent and tension cracks in the middle third developed rapidly again, and the yield point of the horizontal reinforcement was soon passed. The failure caused crushing of concrete at the top of the beam above the crack. The ultimate load was 220 500 lb.

Beam 224-2. The first crack appeared at 35 000 lb. at center about 2 in. high. At the load of 50 000 lb. seven cracks were observed in which the first diagonal cracks appeared 14 in. south from the S.E.L. was included. At the load of 80 000 lb. this diagonal crack extended about the middle height of the web of the beam.

After the applied load reached 90 000 lb. the development of tension cracks seemed to stop and at the load of 105 000 lb. many diagonal cracks were observed at both ends of the beam.

The beam failed by tension at the load of 218 000 lb. followed immediately by the crushing of the concrete at the top of the beam. The load dropped to 151 400 lb.

C. Rectangular beams with 11 in. spacing of the stirrups. Beam 225-1. For this beam the whitewash made of neat cement and lime was so thick that it was very hard to find the cracks.

Five small cracks, most of them about 1 in. high, appeared between or a few inches outside the load point at the load of 35 000 lb.

Until the load of 50 000 lb. was reached these cracks lengthened and became about 10 in. high and four new cracks also appeared.

A diagonal tension crack was observed at the load of 80 000 lb. in the S.E. side. At the load of 200 000 lb. the maximum opening of the diagonal tension crack in the N.W. side was about 0.01 in.

At the load of 229 300 lb. tensile stress in the horizontal steel passed the yield point and the crack opening became 0.09 in. Crushing of concrete at the top of the beam soon followed.

Beam 225-2. Four very small tension cracks were observed at the load of 35 000 lb. at the center of the span in east side and also under the load points. Up to the load of 50 000 lb. ten new tension cracks distributed over the total length of the beam were observed.

The first diagonal cracks appeared at the load of 70 000 lb. on S.E. side;at the load of 80 000 lb. many comparatively large diagonal tension cracks appeared near the both ends.

At the load of 200 000 lb. the maximum opening of the diagonal tension cracks at both ends was 0.02 in.

The tensile stress in the horizontal steel passed the yield point at the load of 214 250 lb. followed by compression failure in the concrete at the top of the beam and the load dropped to 184 000 lb.

19. Rectangular Beams with Vertical Stirrups.- The first diagonal tension crack usually was observed at a load from 50 000 to 80 000 lb. and in most cases these cracks are short, starting from one stirrup and ending at the next stirrup. As the vertical stirrups and horizontal bars kept the crack from widening, there were no crack openings larger than 0.02 inches. However, there were many diagonal cracks developed as the load increased, and some of them started from near the support of the beam toward the load point.

The phenomena of the action of the beams may be considered in three stages.

(1) Tension crack stage. Small tension cracks appeared in the midspan or near the load point at loads between 25 000 and 50 000 lb. These tension cracks developed until the diagonal tension cracks were observed on both sides at the loads between 80 000 and 120 000 lb. After this load was reached the development of the tension cracks became very slow, and diagonal tension cracks appeared quickly and lengthened as the load increased.

(2) Diagonal tension cracks stage. The first diagonal tension cracks usually appeared at a load from 60 000 lb. to 80 000 lb. At the load of 80 000 to 120 000 lb. diagonal tension cracks were observed on each side of the beam. From the first appearance at this load the cracks were very long and the length of many of them were more than 20 in. and developed rapidly.

Except under the load point, cracks however did not go up beyond the two-thirds of the total beam height. The angles between cracks and horizontal reinforcement are almost equal and about 50 degrees. Such a large angle compared with the angles of the diagonal cracks in the beam without web reinforcement may be due to the fact that vertical shearing stress is transferred by the web reinforcement.

Some of these cracks first were observed above the middle height of the web inclining upward; the lower part of the cracks were not observed until a few thousand pounds additional load was applied.

The first cracks appearing were usually those nearest the load point, and the later cracks at increased load appeared near the support. New cracks sometimes, however, were seen between these previous cracks as the load increased.

After the diagonal tension cracks prevailed, the way of distribution of load may change, resembling that of a load at many points, and the deflection at the center may be relieved to some extent.

These phenomena continued until the failure stage was reached.

(3) The failure stage. The failure of the beam will come in the weakest point in the beam, either tensile stress in the horizontal bar; compressive stress or vertical shearing stress on the concrete at the top or vertical tensile stress and bond stress in the concrete at the bottom.

Most of the beams tested, failed by tension at first. After a load from 210 000 lb. to 220 000 lb. tension cracks in the mid-span became suddenly wide, and went up very high to the upper part of the web and crack opening became about 0.1 in. to 0.08 in. showing the yield point had passed.

At the same time, diagonal tension cracks seemed to be suddenly stopping increasing in width. Final failure came by compression in the concrete, the resisting area becoming very small and carrying load decreased suddenly.

20. Stress in Stirrups. Diagram p.137 shows the unit stress in each stirrup. The amount of stress transferred by the stirrups largely depends upon the cracks opened across them, therefore the stress distribution in every stirrup is not uniform.

As usual cracks are wide near the horizontal reinforcement, stresses developed near the bottom are the highest, and their intensity is decreased toward the top of the beam.

The stirrups nearest the support carry a very small amount of stress, and even compressive stress, caused by concentrated reaction and the stiffness of the beam was observed. The amount of compressive stress in the stirrup varied as the distance from the support.

The stirrups nearest the load points also show compression at the lower load. However, after the cracks crossed over the stirrup they carried large tensile stresses.

The maximum stress in the stirrups was usually observed at the middle position after the load of 150 000 lb. of the outer third of the beam as the cracks develop most there.

Table II shows the maximum unit stress in each side of the beams at the load of 200 000 lb. It should be noted that the beams did not fail by the diagonal tension.

TABLE II.

Maximum Unit Stress and total Stress in the Stirrups at the load of 200 000 lb.

Beam No.		W-side			E-side		
		Gage No.	Unit Stress	Stress in Stirrups	Gage No.	Unit Stress	Stress in Stirrups
223-1	N	64 & 74	30 000 lb/in²	3310 lb.	79	31 000 lb/in²	3420 lb.
	S	34	34 000	3750	39	32 000	3530
223-2	N	75	35 500	3920	79	30 000	3310
	S	32	25 500	2820	29	32 000	3530
224-1	N	75	30 000	5900	79	26 000	5110
	S	14 & 24	32 000	6280	29	28 000	5500
224-2	N	75	30 000	5900	88	27 000	5300
	S	24	34 000	6680	29	29 000	5700
228-1	N	74	30 000	9200	68	27 000	8280
	S	14	23 000	7060	28	24 000	7360
228-2	N	74	31 000	9510	79	28 000	8590
	S	14	28 000	8590	19	27 000	8280

21. Effect of Spacing of the Stirrups. The first diagonal cracks usually appeared at the load of 80 000 lb. In one beam the first diagonal cracks were noted at 50 000 lb. and 70 000 lb.

There was no difference in load at which the first diagonal crack was noticed in the beams with different spacings of the stirrups.

The beams with larger spacings of stirrups have wide crack openings at the middle of the spacings. The maximum crack opening in the beams with 4 in. spacing of stirrups was .01 in. and in the beams with 11 in. spacing of stirrups was .02 in. However, the widths of cracks on the stirrups were almost the same with all these beams.

The beams with closer spacings of stirrups have a larger number of small cracks than the beams with larger spacings of stirrups. Perhaps they come from wider cracks giving some relief in the internal stresses.

If the spacing is wide, the amount of shearing stresses transferred by stirrups becomes small.

The ratio between the observed maximum stresses S and calculated stresses S_c are shown in the table 12. The total tensile stresses in the stirrups were calculated by the following formula,

$$S_c = \frac{P}{2jd} \frac{s}{A}$$

where P = applied load in lb.

s = spacing or 4 in., 7 in., and 11 in.

A = number of bars used as the stirrups =

S = 5740s for 4 in. spacing

= 10 075s for 7 in. "

= 1578s for 11 in. "

j = .83

TABLE 12.

Ratio between Observed and Calculated Stirrups Stress.

Beam No.		Load 20,000 lb. W		E	
		S	S/S_c	S	S/S_c
223-1	N	3310	.577	S 3530	.615
	S	3750	.654	N 3420	.596
223-2	N	3920	.690	S 3530	.615
	S	2820	.487	N 3310	.577
224-1	N	5900	.586	S 5500	.546
	S	6280	.623	N 5110	.507
224-2	N	5900	.586	S 5700	.566
	S	6680	.603	N 5300	.526
225-1	N	9200	.582	S 7360	.466
	S	7060	.447	N 8280	.524
225-2	N	9510	.603	S 8280	.524
	S	8590	.544	N 8590	.544

22. Effectiveness of Vertical Stirrups.- Action of stirrups for the diagonal tension is analyzed in Section 4 and phenomena of test show the general agreement with that analysis.

Additional strength due to stirrups is 40% at the load of failure. The highest stress in the stirrups was observed in the Beam No. 22, 3-2 and was 35 500 lb. However this is not the

maximum effectiveness for no beams failed by diagonal tension.

C. Tee-beams with Vertical Stirrups.

23. Notes of Test.-

(a) Tee-beams with 4-in. spacing of stirrups.-

Beam 226-1.- This beam was dropped from a height of about 7 ft. while handling by crane 7 days after being made.

At the load of 25 000 lbs. no cracks appeared.

The first tension cracks appeared 10 in. south from N.E.L. about 2 in. high at 35 000 lbs. At 50 000 lbs. 7 cracks were distributed between the load points or along the stirrup near the load points.

At the load of 70 000 lbs. three new cracks appeared, two of them on N.W. side.

The first diagonal crack was observed near the middle part of the S.E. side at the load of 80 000 lbs., and at the load of 14 000 lbs. a crack reached 1 in. high in the flange under the S.E.L.

When the load of 250000 lbs. was reached the tension cracks became large and just before the measurements of deformation at this load were started, a big noise with shock was heard and the load dropped to 222 500 lbs. By further loading, the beam showed itself able to carry the load of 262 340 lbs., when the second shock occurred and the load dropped to 240 000 lbs. On the third loading, at the load of 252 000 lbs., tension failure was completed and the load dropped to 154 300 lbs. and the beam could not carry any further load.

Beam 226-2.- In this beam there were many holes under the horizontal reinforcing bars.

The first crack was observed 10 in. south from N.E.L. about 1 in. high at the load of 35000 lbs. Until the load of 50 000 lbs. eight cracks were seen and some of them reached about 12 in. high, also a horizontal crack appeared at N.E. side.

At the load of 60 000 lbs. the first diagonal tension crack appeared nearly at the center of the web on the N.E. side. Tension cracks in the midspan of the beam developed and some of them reached the height of 13 in. After the load had reached 80 000 lbs. tension cracks did not lengthen and diagonal tension cracks developed rapidly. Beyond the load of 200 000 lbs. tension cracks opened widely especially after the load of 220 000 lbs. At the load of 240 000 lbs. it was believed that failure might come by the tensile stress passing the yield point of the horizontal bars, and the maximum crack opening at this load was 0.06 in. at the center. The final failure was caused by the crushing of the concrete at the north support. Load dropped to 140 100 lbs. Average bearing stress on the support was 3 110 lbs. per sq. in.

(b) Tee-beams with 7 in. spacing of the stirrups.

Beam 227-1.- At the bottom of the beam the surface was very rough. While putting this beam upon the machine a piece of concrete was broken off the under side of the beam at the support at the south end. A fair bearing , however, was secured.

The first small tension crack occurred at 50 000 lbs. 2 in. north from S.E.L. 1.5 in. high. With the load 60 000 lb had reached two big cracks appeared, one 2-in. south from S.E.L. 12-in. high, and the other 2 in. north from N.E.L. 12 in high.

At the load of 70 000 lbs. the first diagonal crack was developed 10 in. south from S.W.L. up to 10 in high until the nearest stirrup from the load on this side. However, no diagonal tension cracks appeared in the west side until the load of 90 000 lbs. was reached.

Beyond 120 000 lbs. vertical tensile cracks seemed to develop very slowly and diagonal tension cracks extended quickly.

Beyond 200 000 lbs. tension cracks revived and diagonal cracks seemed not to grow and failure was expected to be caused by failure of horizontal reinforcing bars. At the load of 250 000 lbs. diagonal cracks at north side became large and also on the bearing plate of the south end support, chipping cracks were observed accompanied by a small noise and diagonal tension cracks at this south end became large.

The beam carried the load 261 300 lbs, when south end of the beam suddenly broke off without any big noise. After failure it was found that the middle horizontal bar in the bottom layer had been broken near the beginning of the hook. This might be caused by the accident in putting the beam into the machine just before the test.

Beam 227-2.- The first crack appeared at the center of the west side 1 1/2 in. high at the load of 35 000 lb. At the load of 50 000 lbs. nine cracks developed and some of them lengthened more than 14 in.

The first diagonal tension cracks appeared at 80 000 lbs. on both of S.W. side and N.E. side.

After the load of 230 000 lbs. was reached tension cracks in the midspan became large and at 240 000 lbs., tensile stress in the horizontal reinforcement passed their yield point, and tension cracks opened wider. However, the beam carried the ultimate load of 268 600 lbs. Then with a shock and a noise the load dropped to 238 600 lbs. immediately. Upon the application of the further loading, this beam could carry the load of 266 300 lbs. when the secondary failure came on the north end of the beam caused by the crushing stress in the concrete on the support accompanied by the compression failure at the top. Load dropped to 224 300 lbs.

(c) Tee-beams with 11 in. Spacing of Stirrups.

Beam 228-1.- At the load of 50 000 lbs. five cracks appeared suddenly; one of them at the center of the beam on east side 10 in. high, the rest, along the nearest stirrups from the load points, all of them about 14 in. high.

At the load of 60 000 lbs nine new cracks appeared distributed all over the beam from 8 in. to 12 in. high.

The first diagonal tension crack appeared at the south side at the load of 70 000 lbs.

Beyond the load of 120 000 lbs. the tension cracks did not lengthen and diagonal tension cracks developed very greatly

At the load of 220 000 lbs., the largest diagonal tension crack at the N.E.side opened .0026 in.

Beyond the load of 240 000 lbs. tension cracks became larger rather quickly. A noise was heard at the load of 260 900 lbs. and tension cracks in the midspan opened wide followed by another large vertical crack and horizontal crack. The load dropped to 241 200 lbs. The beam carried an ultimate load of 264 200 lbs. and then crushing failure at the north end on the support accompanied by diagonal tension failure completed the final failure of the beam and the load dropped to 146 400 lbs.

Beam 228-2.- At the load of 35 000 lbs. the first two cracks were observed at E side: one 2 in. south from S.E.L. under the stirrups and the other 2 in. north from N.E.L. along the stirrups 10 in. high.

Six new cracks appeared between and under the load points and most of them extended about 13 in. high.

At 80 000 lbs. the first diagonal tension cracks started to appear at N.E.side. At the load of 90 000 lbs. a big diagonal tension crack appeared at S.E.side, and at the load of 105 000 lbs., a crack was found even in the flange in S.W. side.

At the load of 250 000 lbs. diagonal tension cracks became so wide as 0.0510.

At the load of 260 000 lbs. tension cracks near the north side load point became very large and the opening of the crack was 0.06 in. at 10 in. high from the bottom and at the bottom under the N.W.L. was 0.1 in. Load dropped until

235 400 lbs. Upon further loading the beam failed at a load of 255 200 lbs. caused by compressive stress in the concrete at the top. The load dropped to 236 100 lbs.

24. T-shaped beams with vertical stirrups.- The first diagonal tension cracks usually appeared at a load from 60000 to 80 000 lbs. and there were no apparent differences of the amount of load and the way of formation of cracks between T-beams and rectangular beams.

The phenomena of carrying load may also be divided into three stages and the stages are almost the same as in rectangular beams except the last stage. (See Section 19)

The last stage was observed beyond the load of 230000 lbs. to 260 000 lbs, the higher load being caused by the higher position of the neutral axis in this type of beam, and the greater concrete area at the top which was large enough to prevent secondary failure by compression. The carrying load was 19 per cent higher than that of the rectangular beams, and the value of j was also 7 per cent larger.

As the diagonal tension stage was longer in T-beams than rectangular beams diagonal tension cracks appeared more in these beams. The cracks that appeared at these higher loads started from the end of the span either from the bottom, or from the middle height of the web extending both downward and upward. Some of them formed curves having a smaller radius of curvature than those occuring at the lower loads, the latter being as nearly straight and making an angle of about 50 degrees with the horizontal reinforcement.

The reaction at the support was so great that most of the secondary failures took place at the bearing on the support. The cracks that appeared at this stage showed larger angles than the cracks that appeared in the former stages.

The tension cracks also extended so high that some of the beams failed by secondary crushing at the top of the middle of the span.

25. Stress in Stirrups.- Diagrams show the unit stress and total load of each stirrup.

The stress distribution in stirrups of T-beams is more irregular than in the rectangular beam, and maximum stresses were generally observed at the middle height of the beam. Since the flange is stiffer than the web the effect of arch action is to move the maximum stress line up a little. Strain gage readings on the stirrups near the supports showed that ~~these~~ the nearest stirrups, while they had no cracks across the stirrups, stirrups were carrying compression during the greater part of the test. This was due to the stiffness of the flange and concentrated shear at the support.

The stirrups nearest the load points show sometimes compression at the lower loads. However, as the vertical cracks along the stirrups appeared at a load of about 50 000 lb., the stirrups carried a great deal of tensile stress.

The stresses in stirrups in T-beams were somewhat larger than the stresses in stirrups in rectangular beams. A single large crack is more dangerous than many cracks crowded together. At the load of 26 000 lb. the gage line No.14 in the beam 228-1 showed the unit stress of 44 500 lbs.

The table shows the maximum unit stress on each side of the beams at the load 200 000 lb. It should be borne in mind that the beams did not fail by diagonal tension.

TABLE 13.

Maximum Unit Stress and Total Stress in the Stirrups at the Load of 200000 lb.

Beam	No.	W - side			E - side		
		Gage No.	Unit Stress	Stress in Stirrups	Gage No.	Unit Stress	Stress in Stirrup
			lb/in²	lb		lb/in²	lb.
226-1	N - end	63	32 500	3 590	69	30 000	3 310
	S - end	23	27 000	2 980	19	30 000	3 310
226-2	N	53	31 000	3 420	78	26 000	2 870
	S	13	28 000	3 090	19	31 000	3 430
227-1	N	84	35 000	6 870	89	33 000	6 480
	S	14	25 000	4 910	39	39 000	7 660
227-2	N	73	31 000	6 090	88	32 500	6 380
	S	25	23 000	4 520	20	30 500	5 990
228-1	N	63	24 500	7 520	68 & 80	19 000	5 830
	S	14	31 000	9 520	19	29 000	8 900
228-2	N	63	29 000	8 900	80	23 000	7 060
	S	23	29 500	9 050	29	33 000	10 150

26. Position of Neutral Axis.- Analysis for the location of neutral axis in the T-beam using the parabolic formula may be shown as follows:

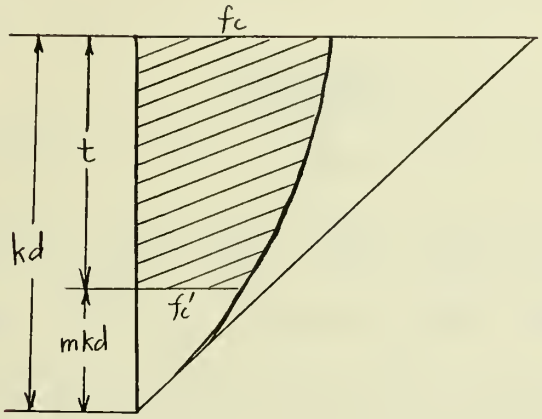


Fig.10.

$$y = (1-\tfrac{1}{2}q)\varepsilon_c$$

$$\text{Area} = \tfrac{1}{2}(1-\tfrac{1}{3}q)\varepsilon_c kd E_c$$

$$f_c = (1-\tfrac{1}{2}q) E_c \varepsilon_c$$

$$f_c' = (1-\tfrac{1}{2}qm) E_c \varepsilon_c m$$

Now compression in flange = shaded area

$$= \tfrac{1}{2}(1-\tfrac{1}{3}q) E_c \varepsilon_c k(b'-b)d - \tfrac{1}{2}\left[E_c \varepsilon_c (b'-b) d (1-\tfrac{1}{3}qm) km\right]$$

$$= \tfrac{1}{2} E_c \varepsilon_c (b'-b) d \left[(1-\tfrac{1}{3}q) - (1-\tfrac{1}{3}qm) m\right] k$$

Equating horizontal tension and compression.

$$A E_s \varepsilon_s = \tfrac{1}{2}(1-\tfrac{1}{3}q) E_c \varepsilon_c kdb + \frac{(b'-b)}{2} E_c \varepsilon_c d\left[(1-\tfrac{1}{3}q) - (1-\tfrac{1}{3}qm) m\right] k$$

Now $\quad \frac{\varepsilon_s}{1-k} = \frac{\varepsilon_c}{k}$

and put $\quad \frac{A}{bd} = p \quad$ and $\quad \frac{E_s}{E_c} = n$

then $\quad pn(1-k) = \tfrac{1}{2}(1-\tfrac{1}{3}q)k^2 + \frac{b'-b}{2b}\left[(1-\tfrac{1}{3}q) - (1-\tfrac{1}{3}qm) m\right] k^2$

$m = 1 - \frac{t}{kd}$

From these equations

$$k = \frac{-\left[pn + \frac{b'-b}{2b}\left(1 - \frac{2}{3}q\right)\frac{t}{d}\right] + \sqrt{\left[pn + \frac{b'-b}{2b}\left(1 - \frac{2}{3}q\right)\frac{t}{d}\right]^2 + 2\left(1 - \frac{2}{3}q\right)\left(pn - \frac{b'-b}{2b}q\frac{t^2}{d^2}\right)}}{1 - \frac{2}{3}q}$$

The value of k in the different stages of resistance are shown in Table 14.

TABLE 14.

Location of the neutral axis in T-beam

q	.1	.2	.3	.4	.5	.6	.7	.8	.9	.10
k	.312	.309	.307	.306	.306	.306	.305	.305	.304	.304
kd	6.56	6.49	6.45	6.42	6.42	6.42	6.40	6.40	6.39	6.39

As the location of neutral axis is very near the bottom of flange, the formula for j in rectangular beams may apply without any objection.

$$j = 1 - \frac{1}{3}\left(1 + \frac{1}{4}\cdot\frac{q}{3-q}\right)k$$

TABLE 15.

Value of k and j in T-beam

q	.1	.2	.3	.4	.5	.6	.7	.8	.9	.10
k	.312	.309	.307	.306	.306	.306	.305	.305	.304	.304
j	.895	.895	.895	.894	.893	.892	.891	.889	.888	.886

27. Effect of Spacing of the Stirrups.- Variation in the spacing of stirrups did not show any large difference in the load at which the first diagonal cracks appeared. The first crack generally appeared at a load of about 70 000 lb.

The effect of spacing of the stirrups upon the cracks formation is almost the same as that with the rectangular beams.

The beams with larger spacings have wide crack openings, while width on the stirrups are almost same.

The beams with smaller spacing of stirrups have more cracks than that with larger spacings as is the case in rectangular beams.

As the stresses in stirrups depend greatly upon the position of large cracks and since the location of cracks was very irregular in the T-beams the same conclusions on the effect of stirrups in the rectangular beams could not be applied to this type of beam.

The ratio between the observed maximum stress S and calculated stress Sc was obtained in the same way as with rectangular beams using the value of j = .89 as shown in Table 16.

TABLE 16.

Maximum observed and calculated stresses in the stirrups at the load of 200 000 lbs.

		North		South	
		S	S/Sc	S	S/Sc
226-1	W	3590	.671	2980	.557
	E	3310	.619	3310	.619
226-2	W	3420	.639	3090	.578
	E	2870	.536	3430	.641
227-1	W	6870	.734	3910	.524
	E	6480	.692	7660	.818
227-2	W	6090	.650	4520	.483
	E	6380	.681	5990	.640
228-1	W	7520	.512	9520	.649
	E	5830	.397	8900	.605
228-2	W	8900	.606	9050	.616
	E	7060	.481	10150	.690

D. Rectangular Beam with Bent-up Bars.

28. Notes of Test.-

Beam 229-1.- The side faces of this beam were slightly inclined from the vertical. No cracks had appeared at the load of 40 000 lbs. At the load of 50 000 lb. twelve cracks appeared at once, some of them reached 10 in. high. At the load 60 000 lbs. most of these cracks had lengthened and seven new cracks appeared in various parts of the beam even 22.7 in south from the S.W.L. Large diagonal cracks were observed on north side caused probably by the slipping of the bent-up bar. The maximum crack opening on this side was 0.028 in. at the load of 200 000 lbs.

The failure was a tension failure in the horizontal reinforcement, followed by a compression failure at the top of the beam. The ultimate load was 225 400 lbs., decreasing to 181 500 lbs.

Beam 229-2.- At the load of 50000 lbs, the first ten cracks appeared, distributed between points of bent up bars.

At 80 000 lbs. cracks along the bent-up bars were observed at the south side. At the load of 105 000 lbs, many cracks had appeared all over the beam.

At the load of 200 000 lbs. deformation of the concrete under and near the S.E.L. increased rapidly, showing the failure of concrete was near. The beam failed by the crushing of the concrete at the load of 213 700 lbs. and the load dropped to 189 000 lbs.

29. Rectangular beams with Bent-up Bars.- Only two beams of this type were tested. Two bars were bent-up for shear reinforcement, one at 6 in. and the other at 15 in. from the load point.

The diagonal tension cracks were observed at the load of 60 000 lbs. in beam No. 229-1 and 50 000 lbs. in beam No. 229-2. Therefore, it seems safe tosay that diagonal tension cracks in this type of beams may be observed at a somewhat smaller load than in beams with all horizontal bars straight.

In studying the load carrying phenomena it may be better to divide the work into three stages as was done in the beams with vertical stirrups, though the stages are not as definite as the case of the beams with stirrups.

Even in the diagonal tension stage, a few more vertical cracks opened in the middle third of the span.

At the higher load many curved cracks appeared near the end of bent-up bars.

The crack openings were wider than those of beams with vertical stirrups and the maximum width observed was .028 in. at the load of 200 000 lbs. in Beam 229-1.

The last stage started at the load of about 200 000 lbs. Both beams failed by tension in the steel at the center of the span, followed by compression failure at the top.

The ultimate load for Beam No. 229-1 was 225 400 lbs. and for Beam No. 229-2 was 213 700 lbs.

Taking average value of these ultimate loads the average shearing unit stress for these beams is 789 lbs. per sq. in. if the value j is assumed to be .83 as in the other type of rectangular beams.

30. Conclusions.- The foregoing discussion may be summed up in the following conclusions:

I. In deep beams with short span, high diagonal tensile stresses can be developed.

II. In these tests, beams without stirrups carried an average shearing unit stress of 525 lbs. equal to 0.12 fc'.

III. Hooks at the ends of bars are effective in preventing slipping of bars, and it is desirable to have hooks at the ends of the bar even when corrugated bars are used. These hooks are evidently essential to the development of high shearing stresses.

IV. Beams without web reinforcement failed by either (1) shearing or breaking of the concrete at the top near the load point or (2) vertical tension or stripping of the concrete above the reinforcing bars and near the diagonal tension cracks.

V. After diagonal tension cracks developed the tension cracks in the middle which had appeared before did not develop rapidly.

VI. At the final failure stage, when the beam failed by tension in the steel, the diagonal tension cracks did not develop rapidly.

VII. Vertical stirrups are effective in preventing crack widening and also in transfering shearing stress. In the beams tested, the resisting strength was increased at least 40 per cent by the use of vertical stirrups though as the beams failed by tension in the horizontal reinforcement the

full effectiveness was not developed.

VIII. The beams with the closer spacing of stirrups had a larger number of small cracks than the beams with the larger spacing of stirrups.

IX. The beams with the larger spacing of stirrups had wider crack openings midway between stirrups than the beams with the smaller spacings of stirrups.

X. In the rectangular beams, as the spacing of stirrups increased the amount of shearing stress transferred by stirrups decreased slightly.

XI. For stress in stirrups the coefficient C in the formula $S = C \frac{Vs}{Ajd}$ was found to be a little less than 2/3. Therefore, 2/3 can be used for practical purposes even in a deep beam.

XII. The effect of spacing of stirrups in T-beams is almost the same as in rectangular beams. However, the crack formation is more complicated and irregular in the T-beams. Therefore, the position of maximum stress in stirrups is also irregular, and the value is somewhat higher than in rectangular beams.

XIII. The diagonal tensile stress distribution in T-beams was almost the same as in rectangular beams. However, the diagonal tension stage is longer in T-beams than in rectangular beams.

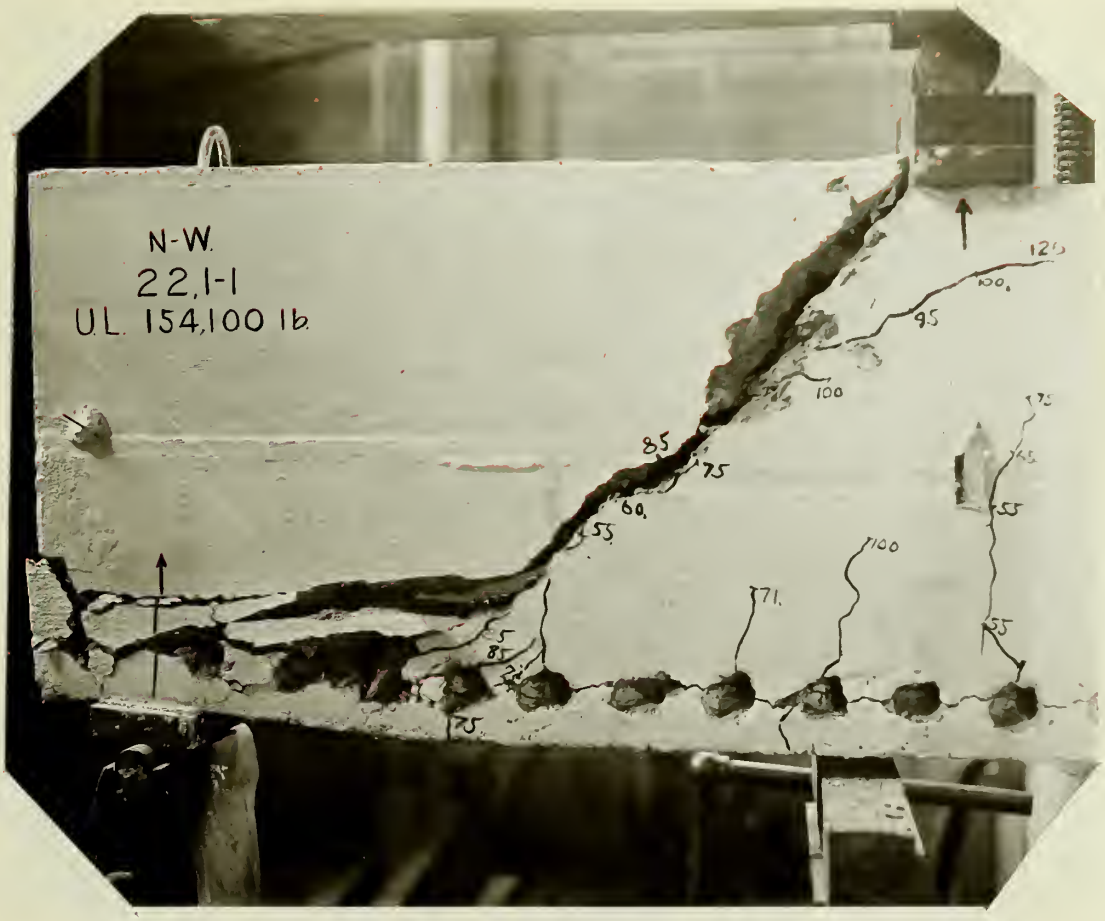
XIV. The T-beams carried an average of 19 per cent higher load than the rectangular beams.

XV. In the T-beams the coefficient C in the formula $s = C\frac{Vs}{Ajd}$ varies more than in the rectangular beams, and in one beam the value of C was considerably greater than 2/3.

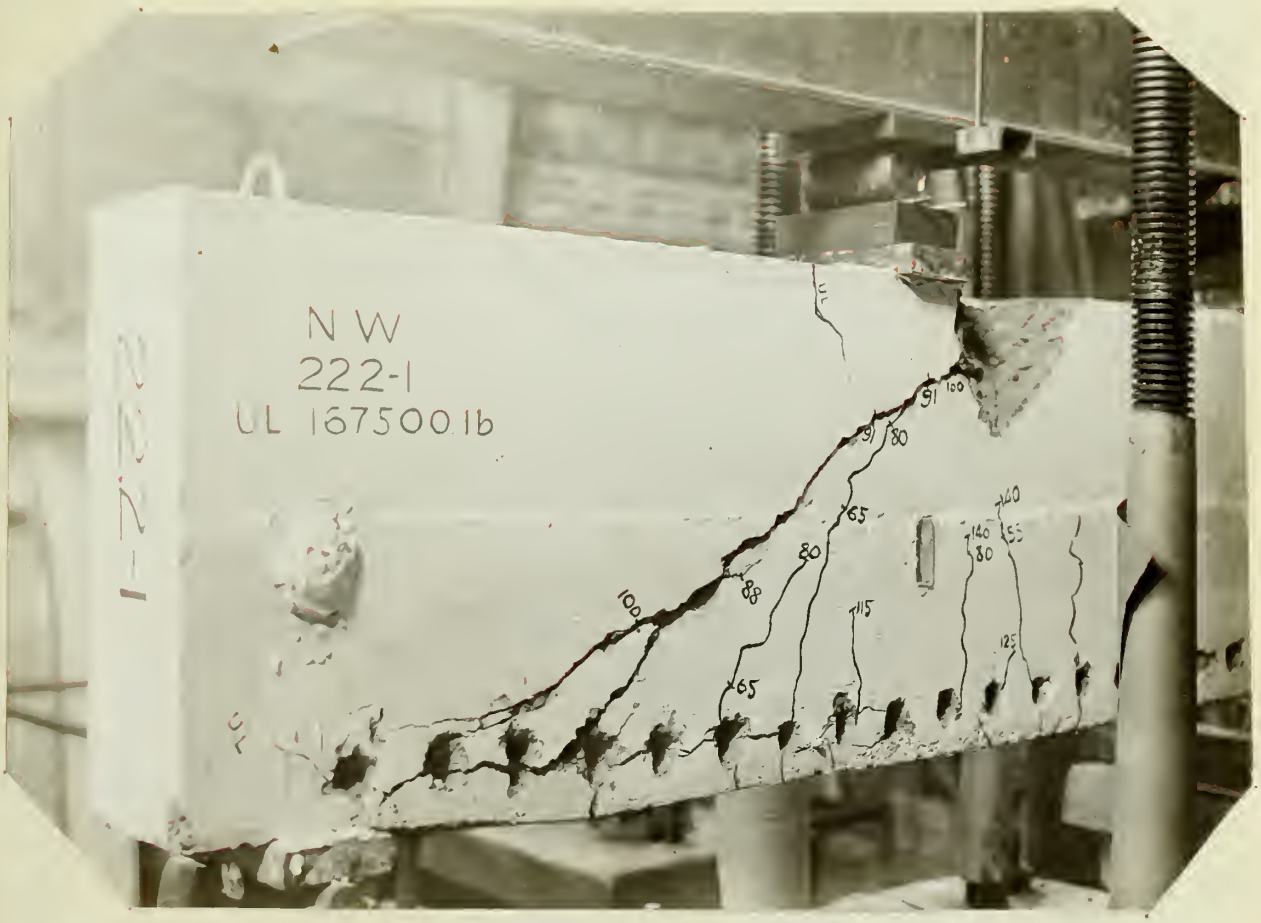
XVI. A concentrated large crack is more dangerous than a number of distributed cracks. Such a single concentrated crack may be caused by non-homogeneity of the beam or by wide spacing of stirrups; the formation of a number of smaller cracks may be produced by homogeneity of material or by well spaced and distributed stirrups.

XVII. Bent-up bars are effective in preventing widening of cracks and also in transferring shearing stress. The resisting strength added by bent-up bars in the one type of beam tested was at least 40 per cent of the ultimate strength of the beams without web reinforcement. The full effect was not developed, since the beam failed by tension in the horizontal reinforcement

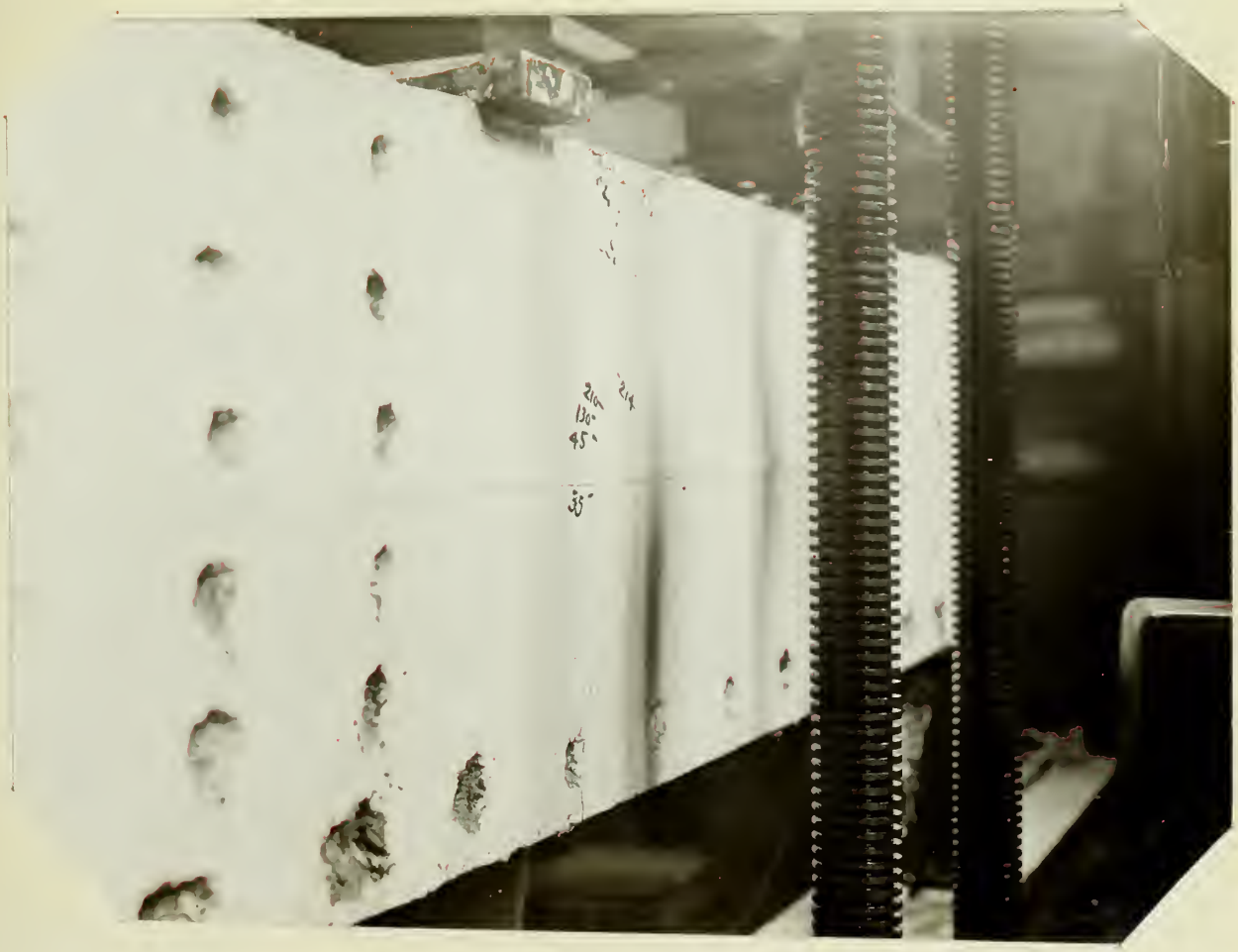
PHOTOGRAPHS AND DIAGRAMS



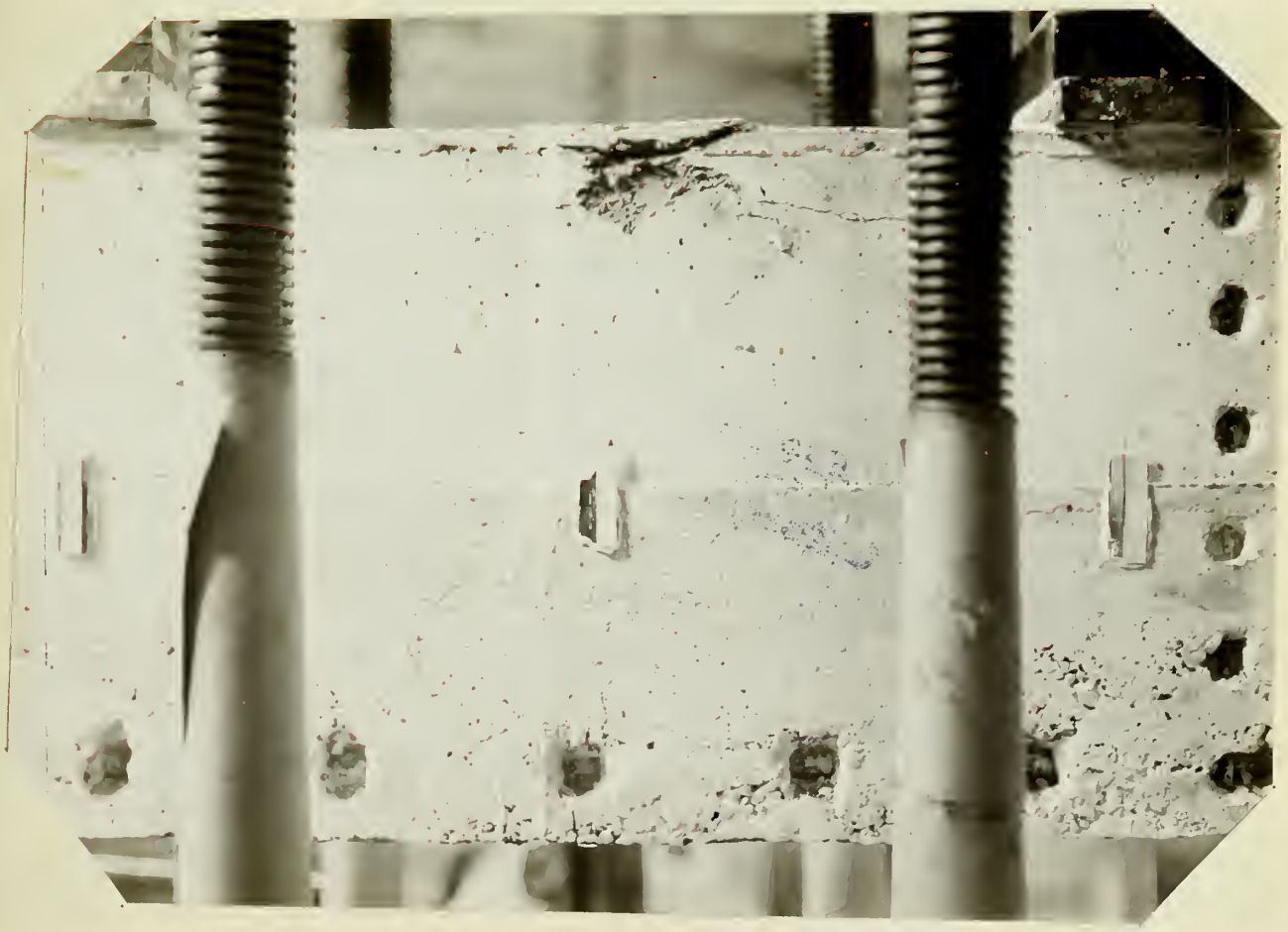
8. View of Beam No. 22,1-1 at failure.



9. View of Beam No. 2,2-1 at failure.

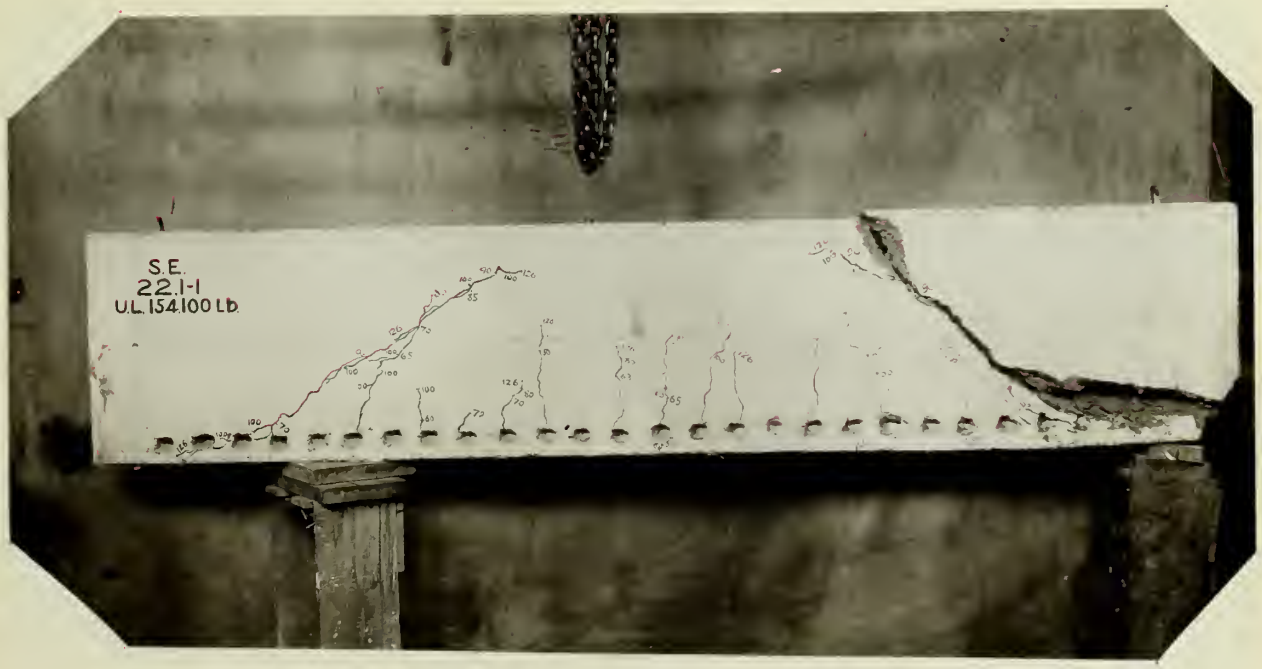


10. View of Beam No. 22,3-1 at failure.

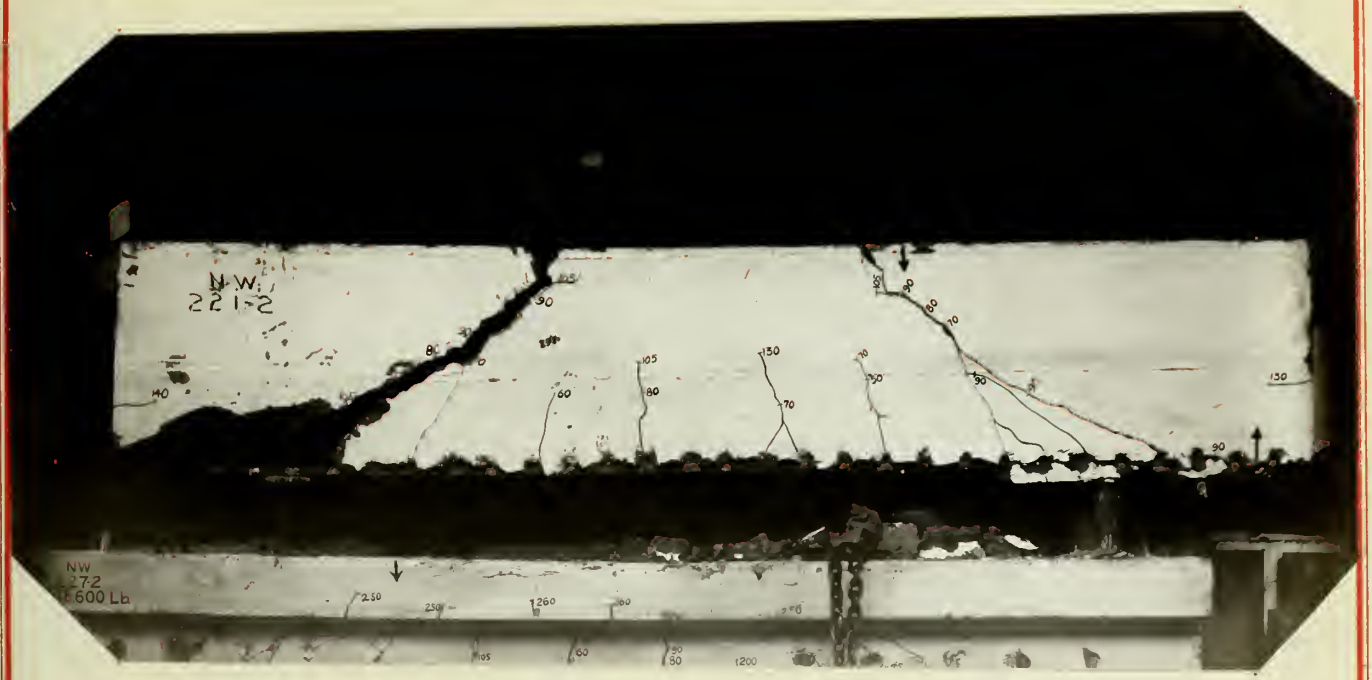


11. View of Beam No. 22, 4-1 at failure.

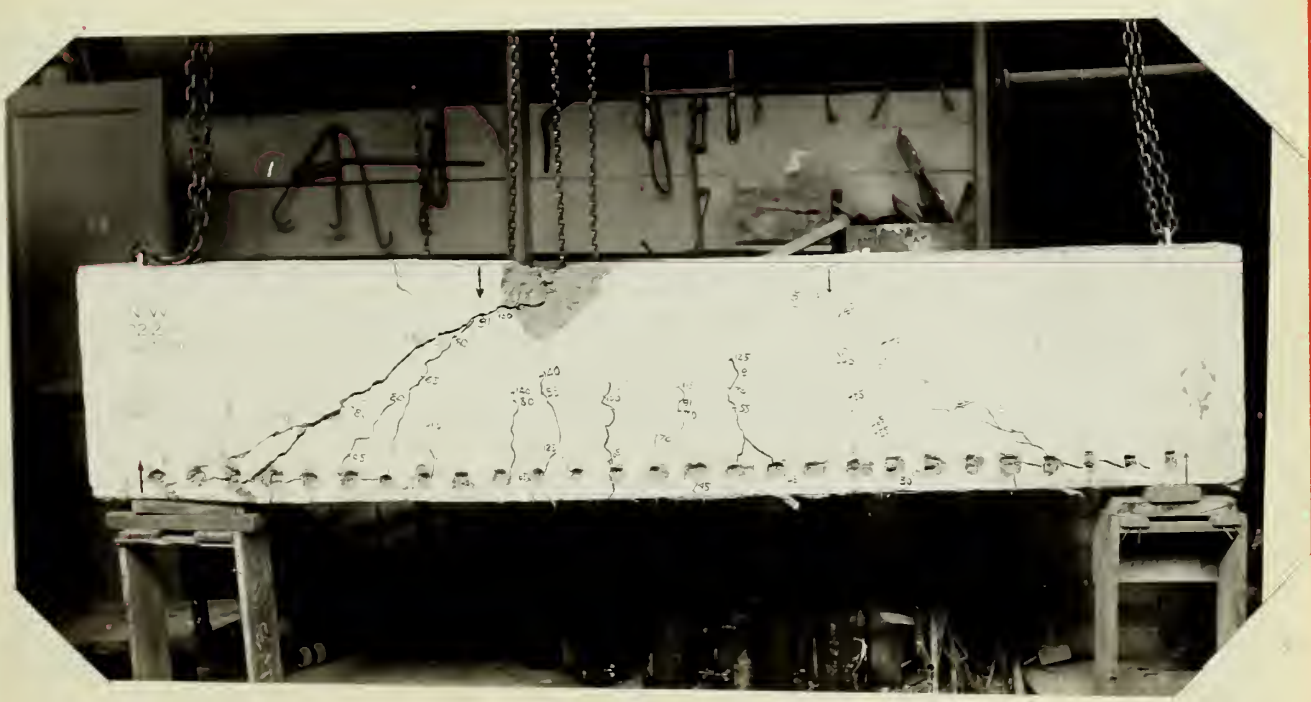
12. View of Beam No. 22,5-1 at failure.



13 and14. Beam No.22,1-1



15 and 16. Beam No. 1-2.



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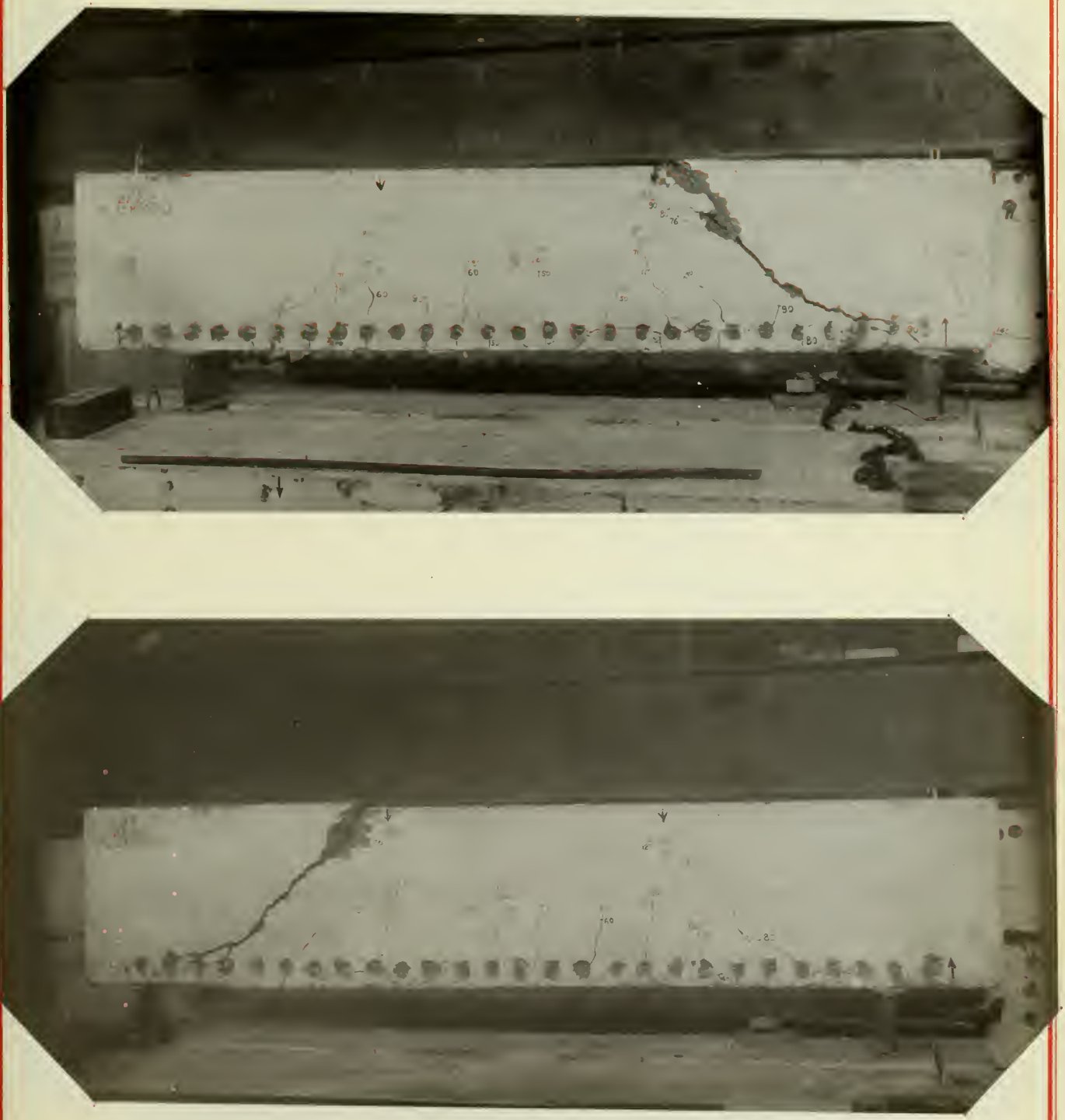

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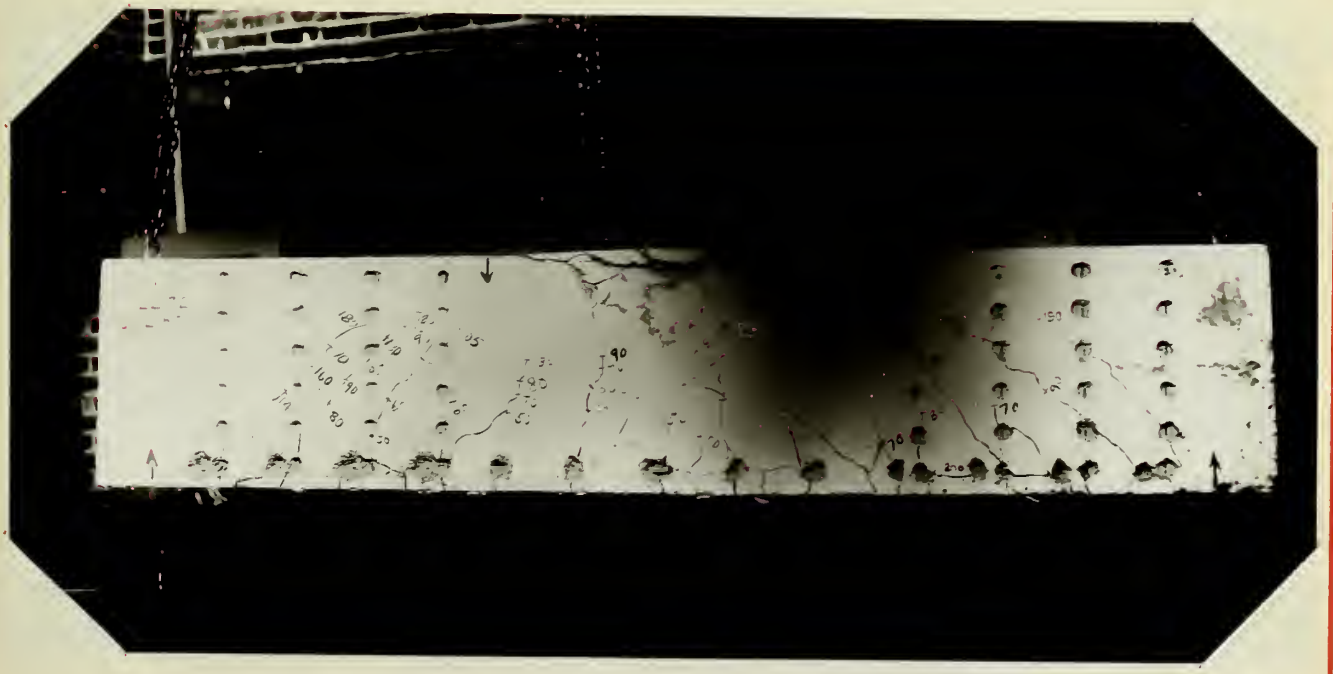
21 and 22 Beam No. 22,3-1.



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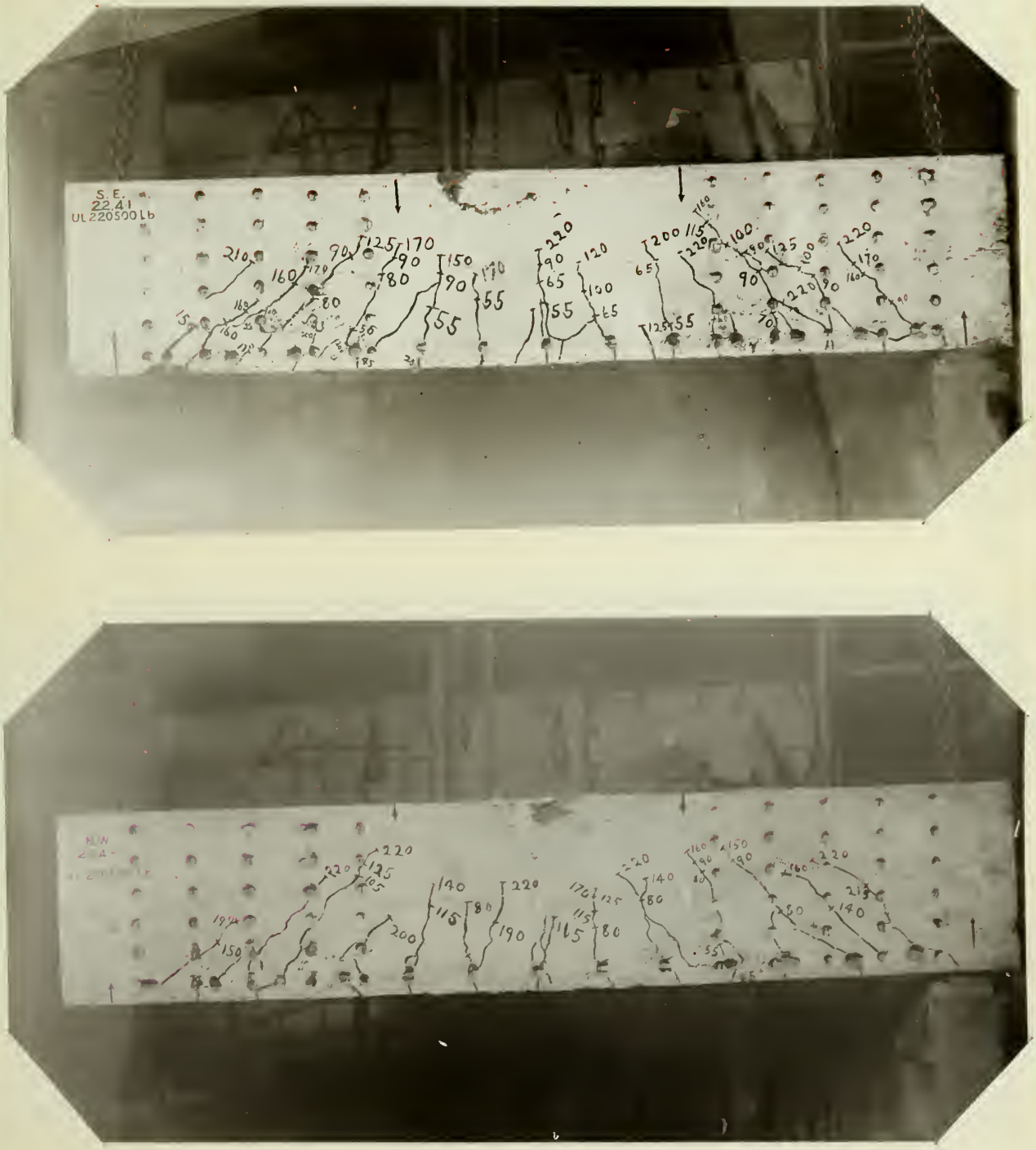

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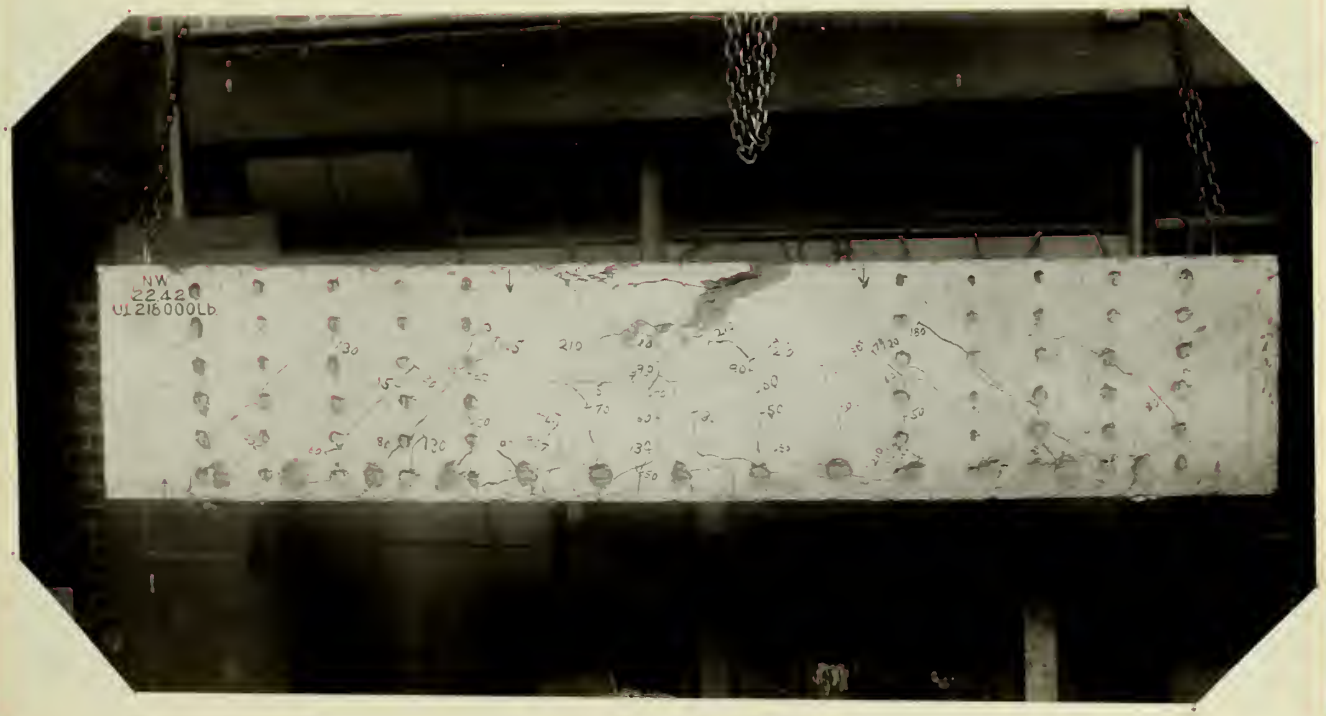

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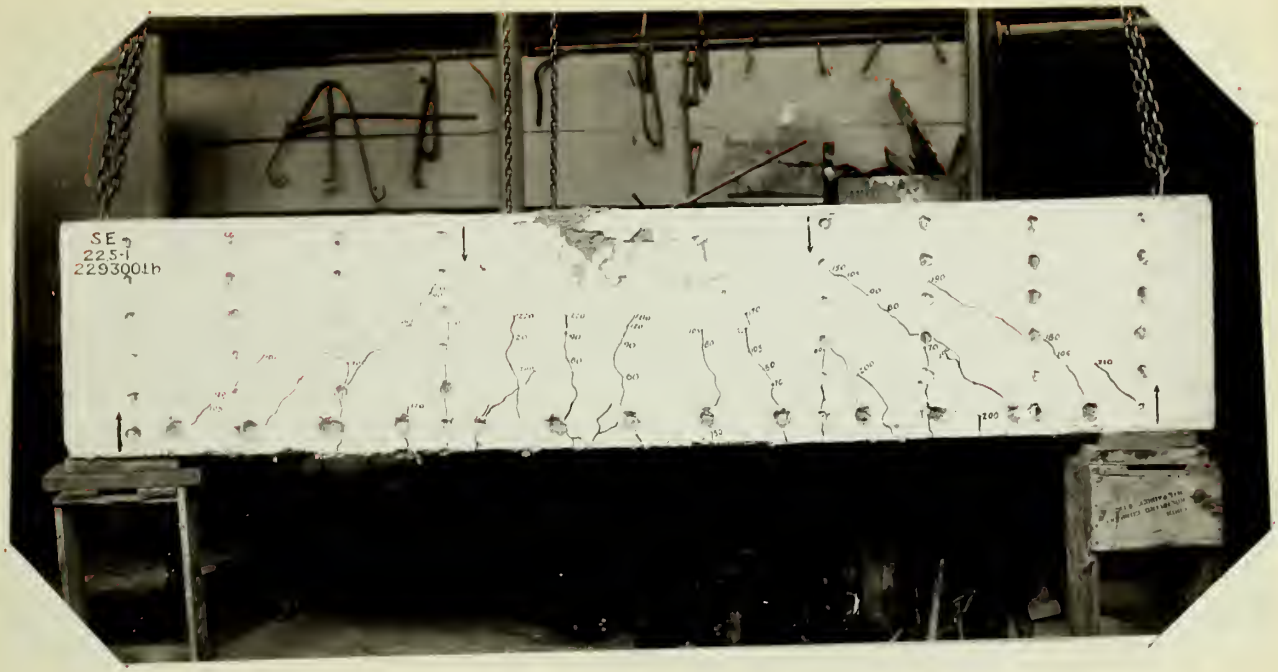

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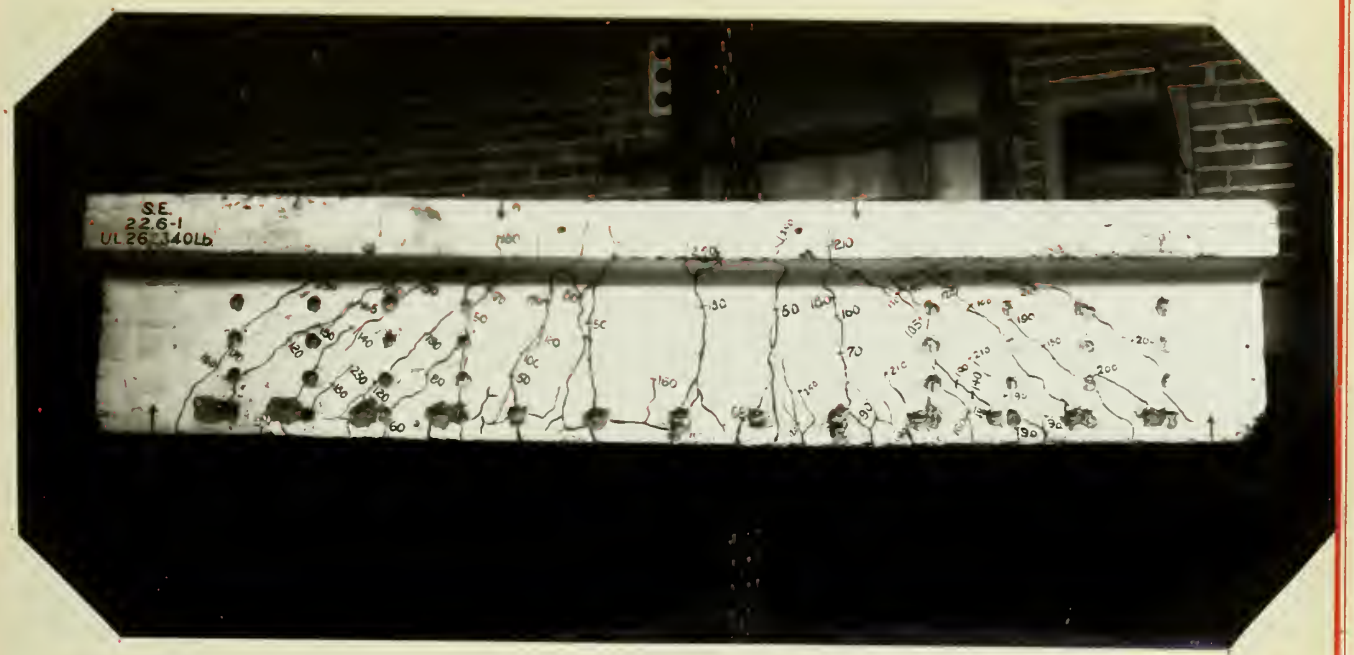
31 and 32 Beam No. 22,5-2.



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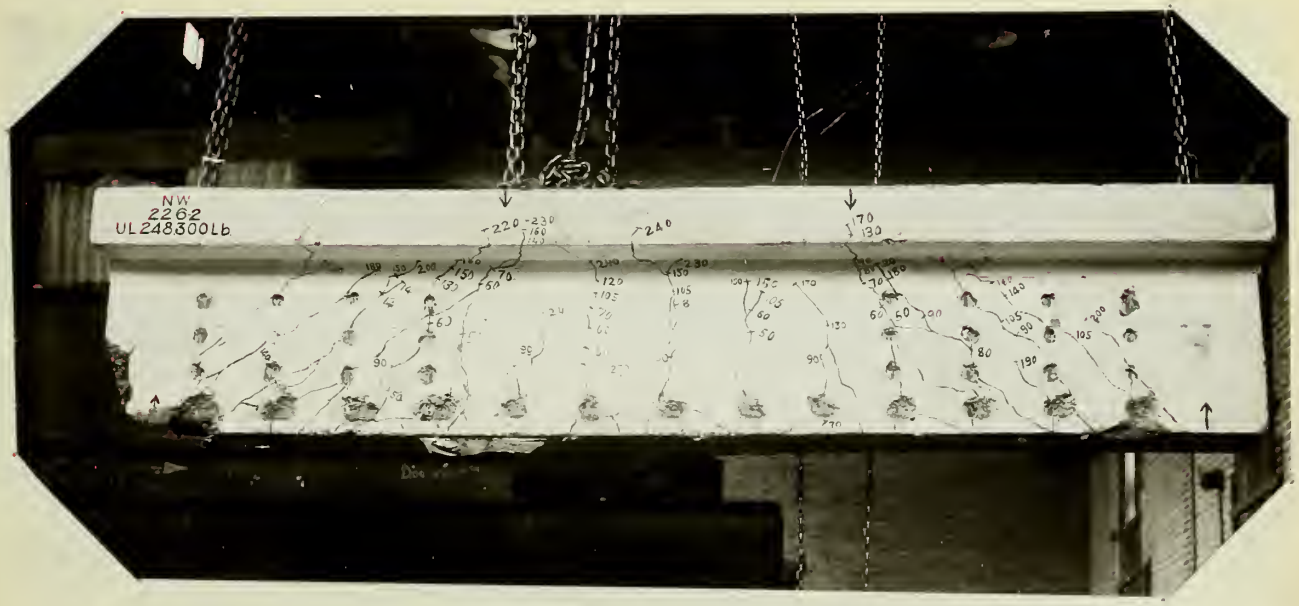

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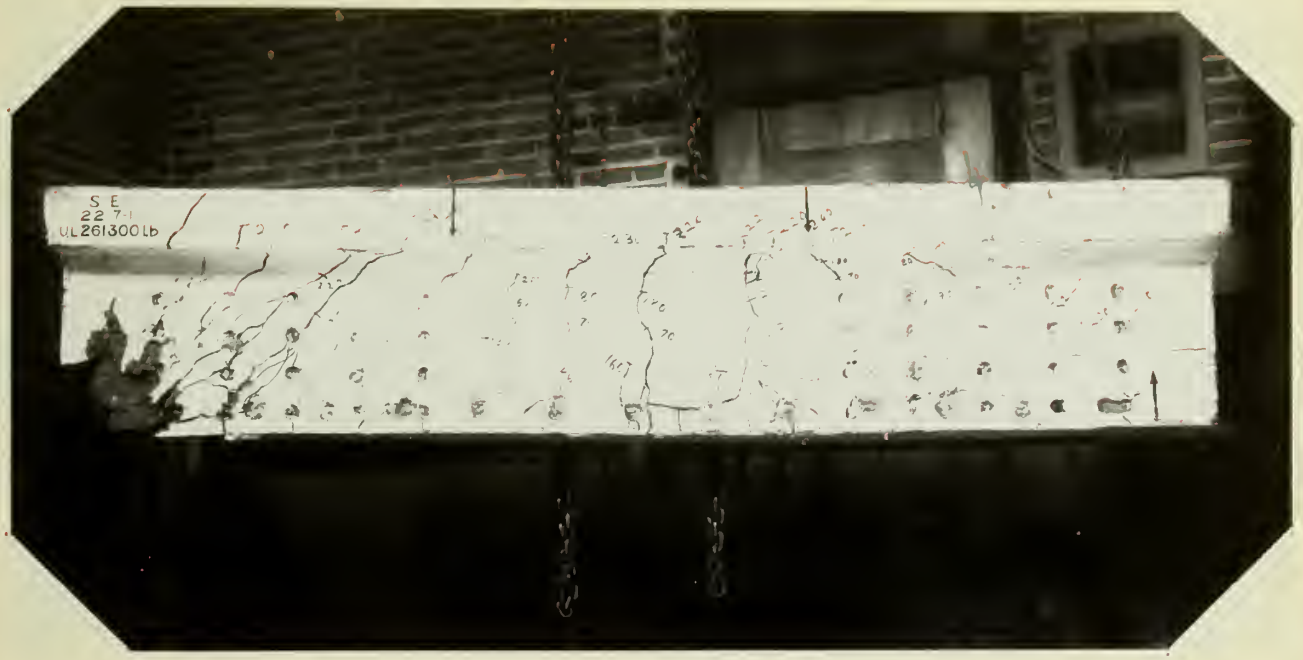

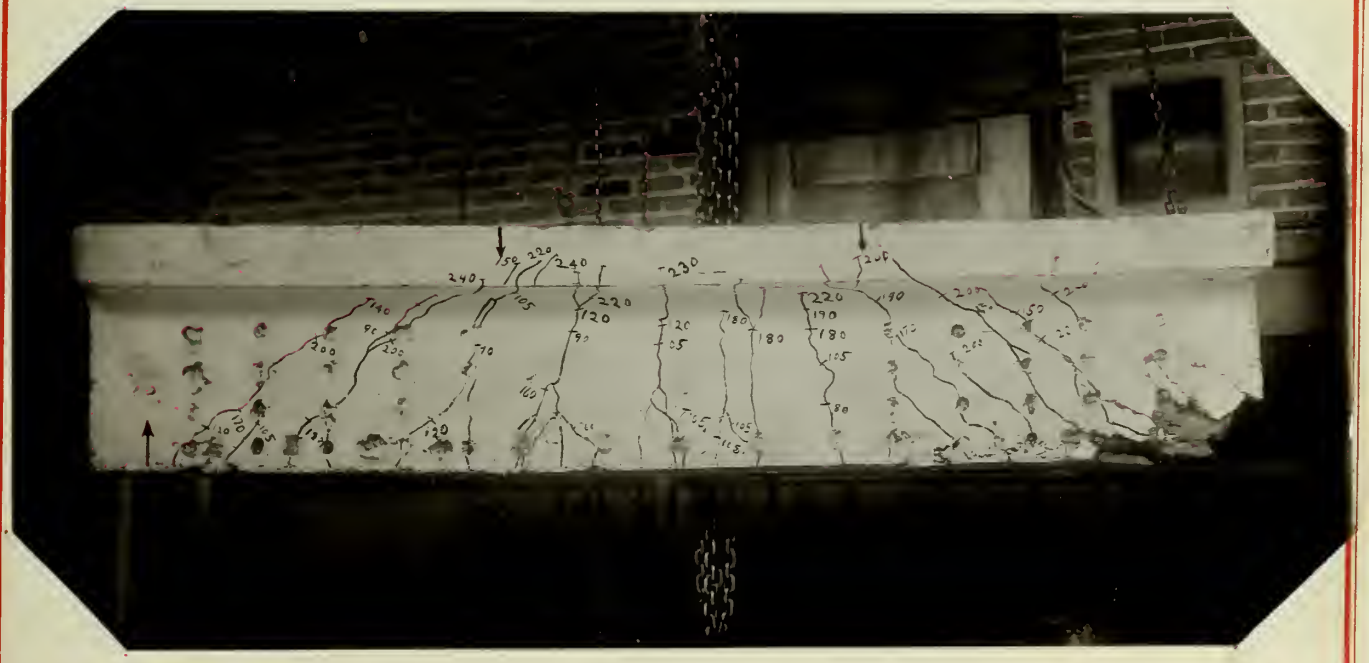

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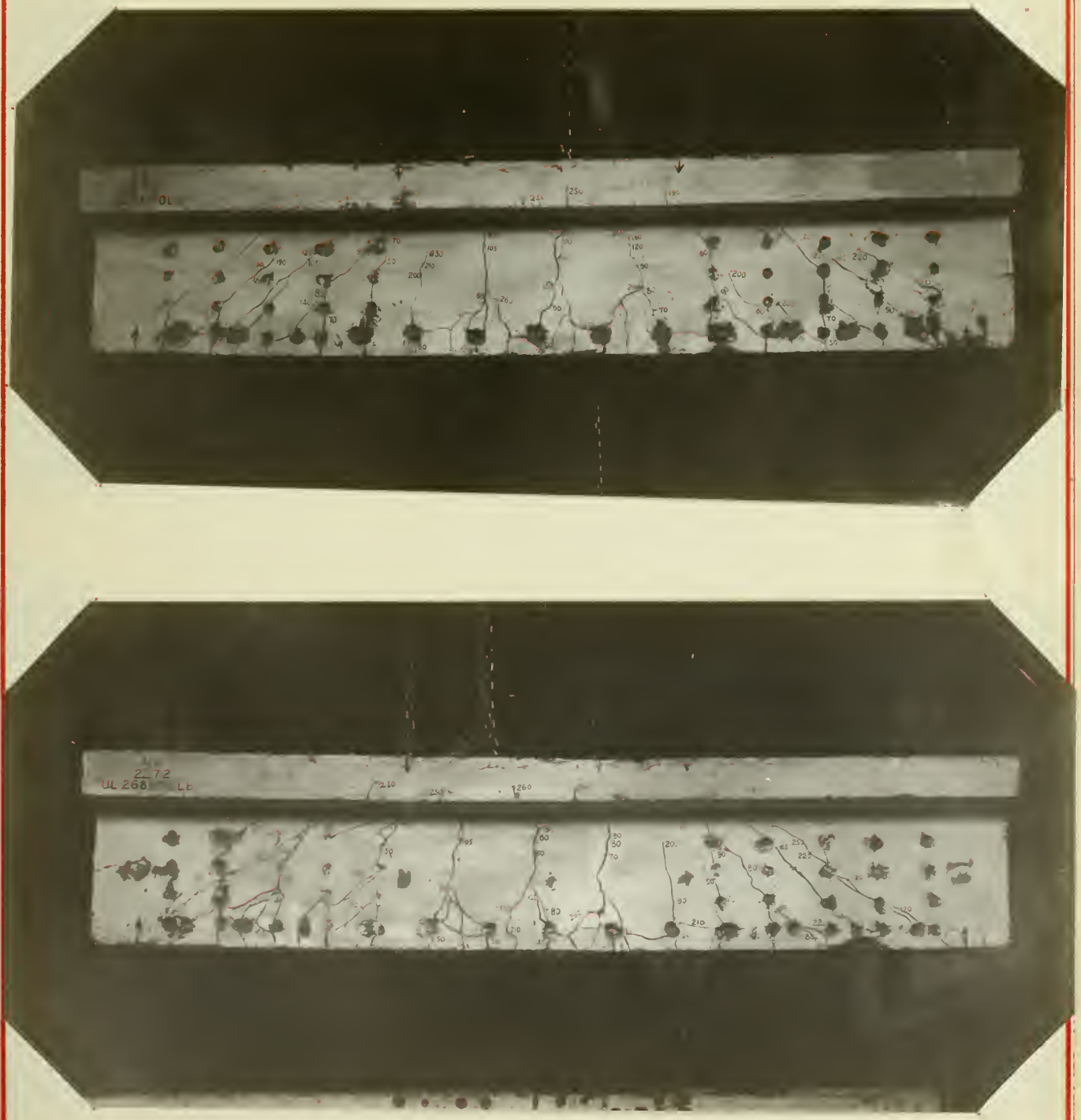

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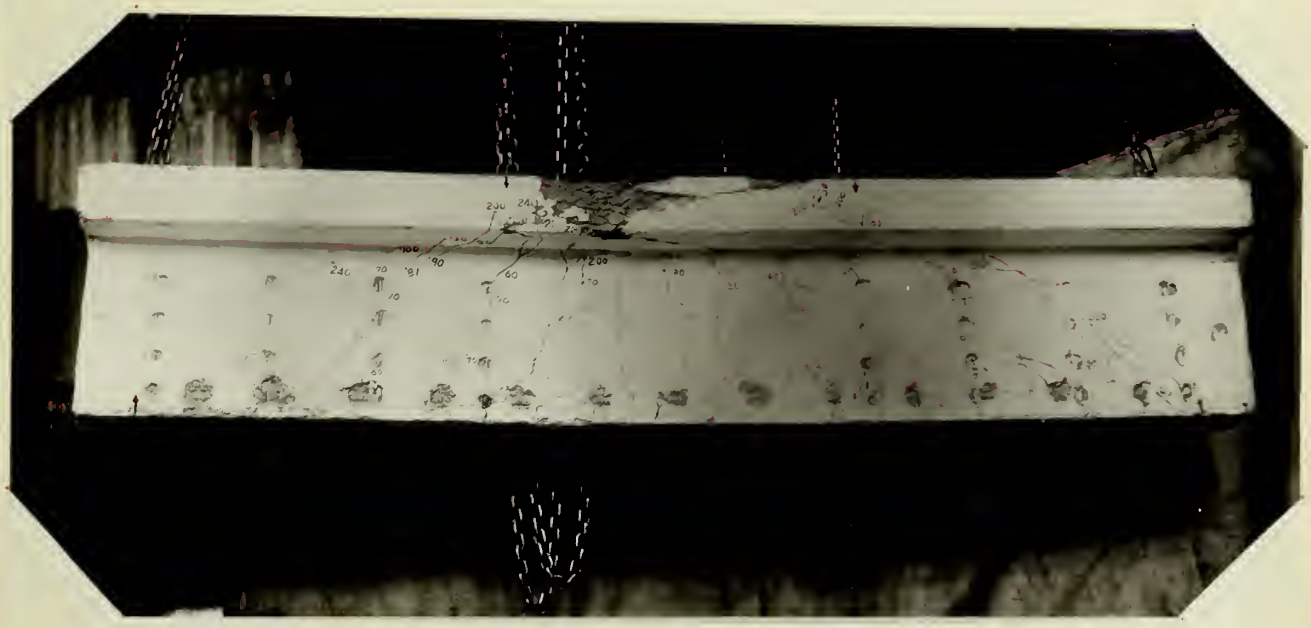
41 and 42 Beam No. 28,8-1



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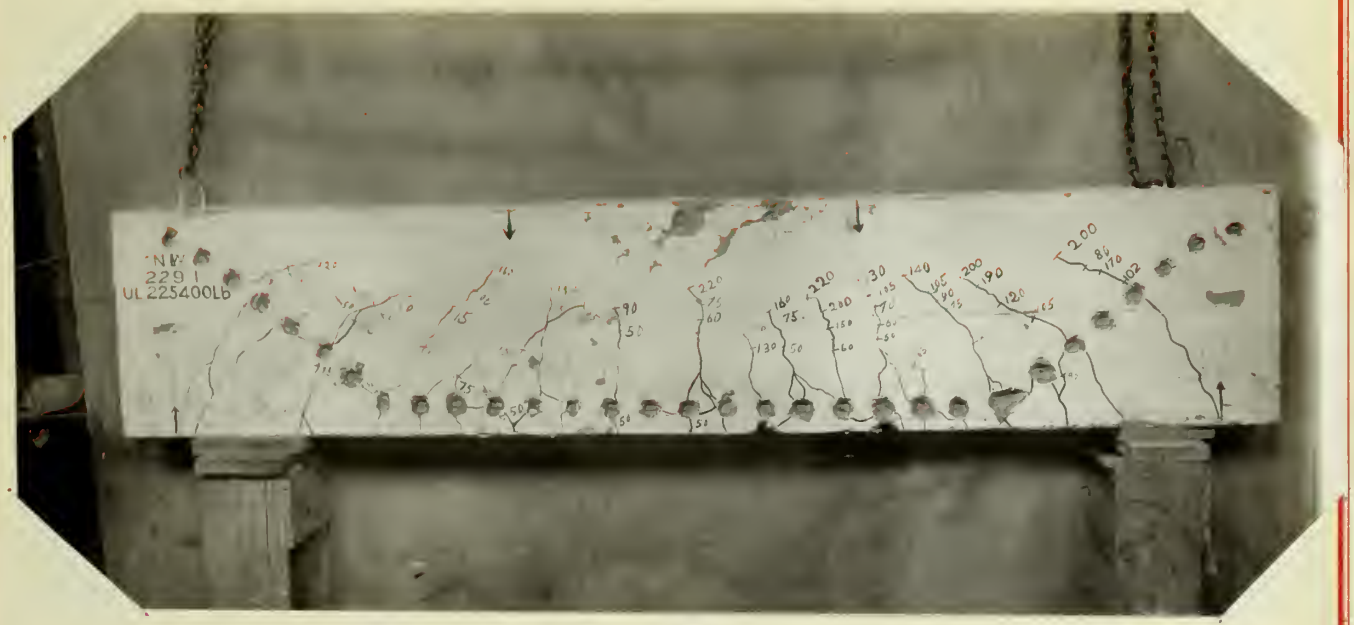

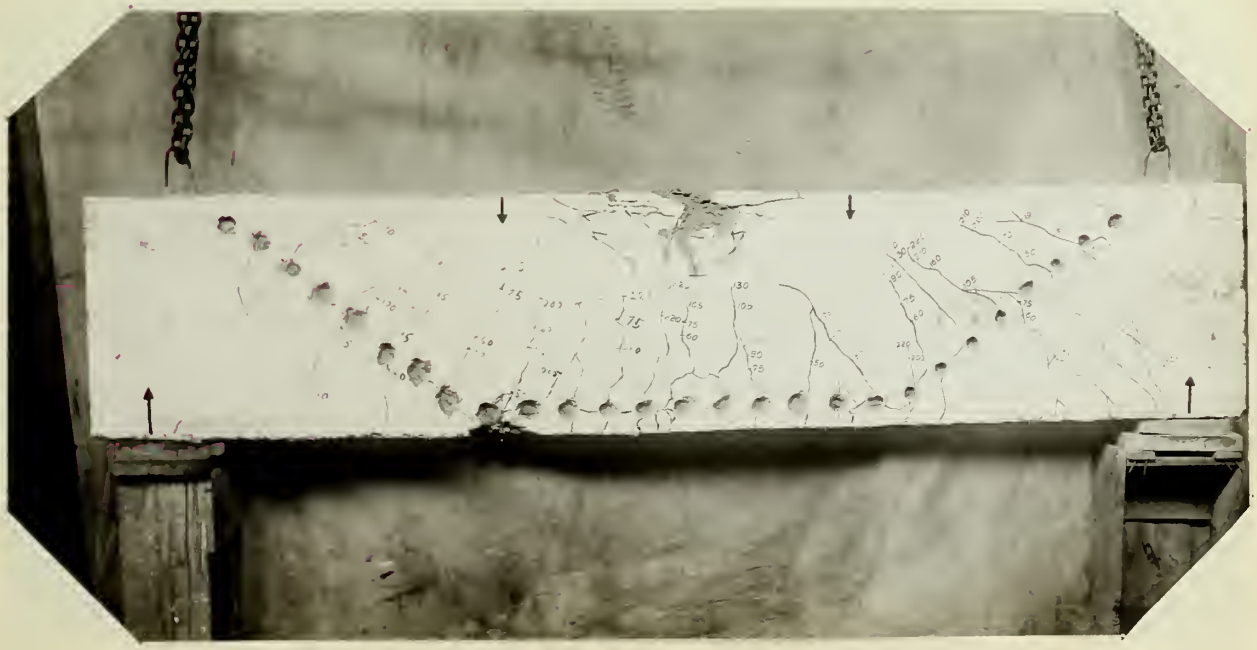

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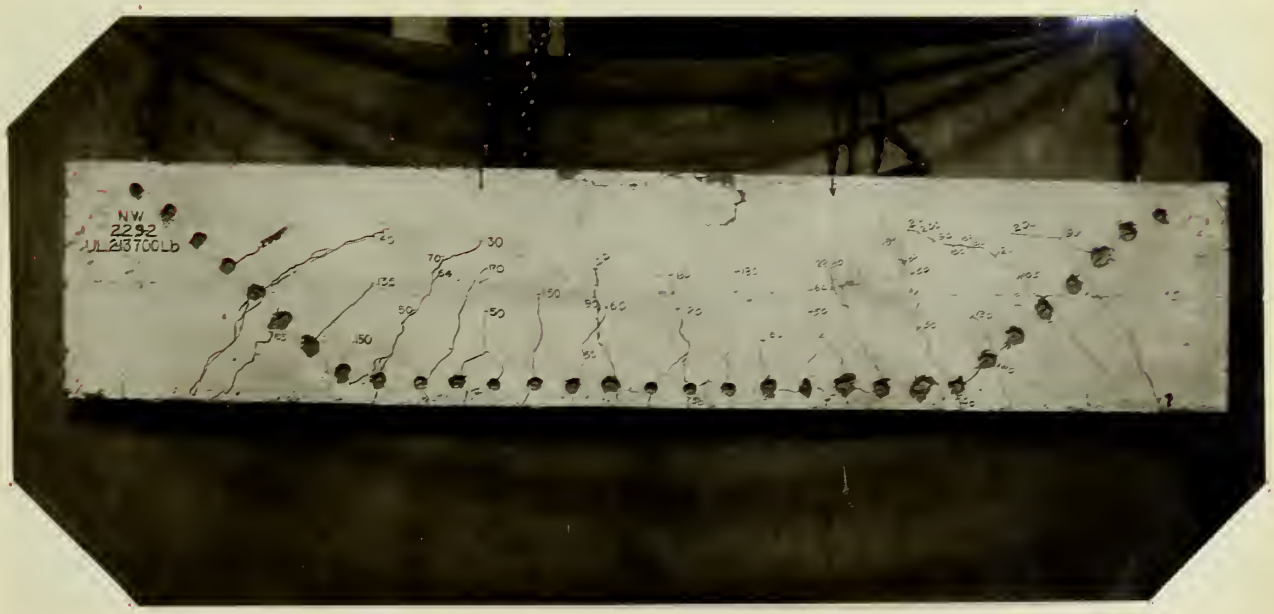

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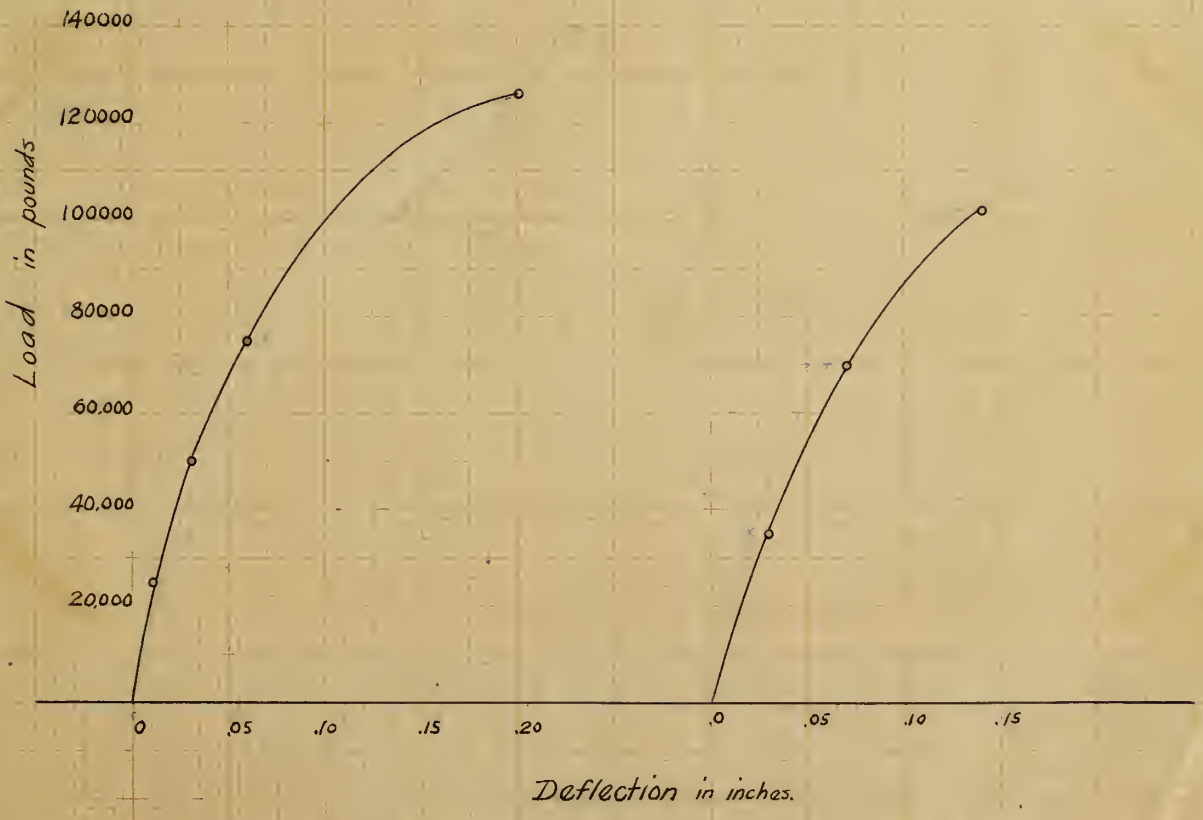

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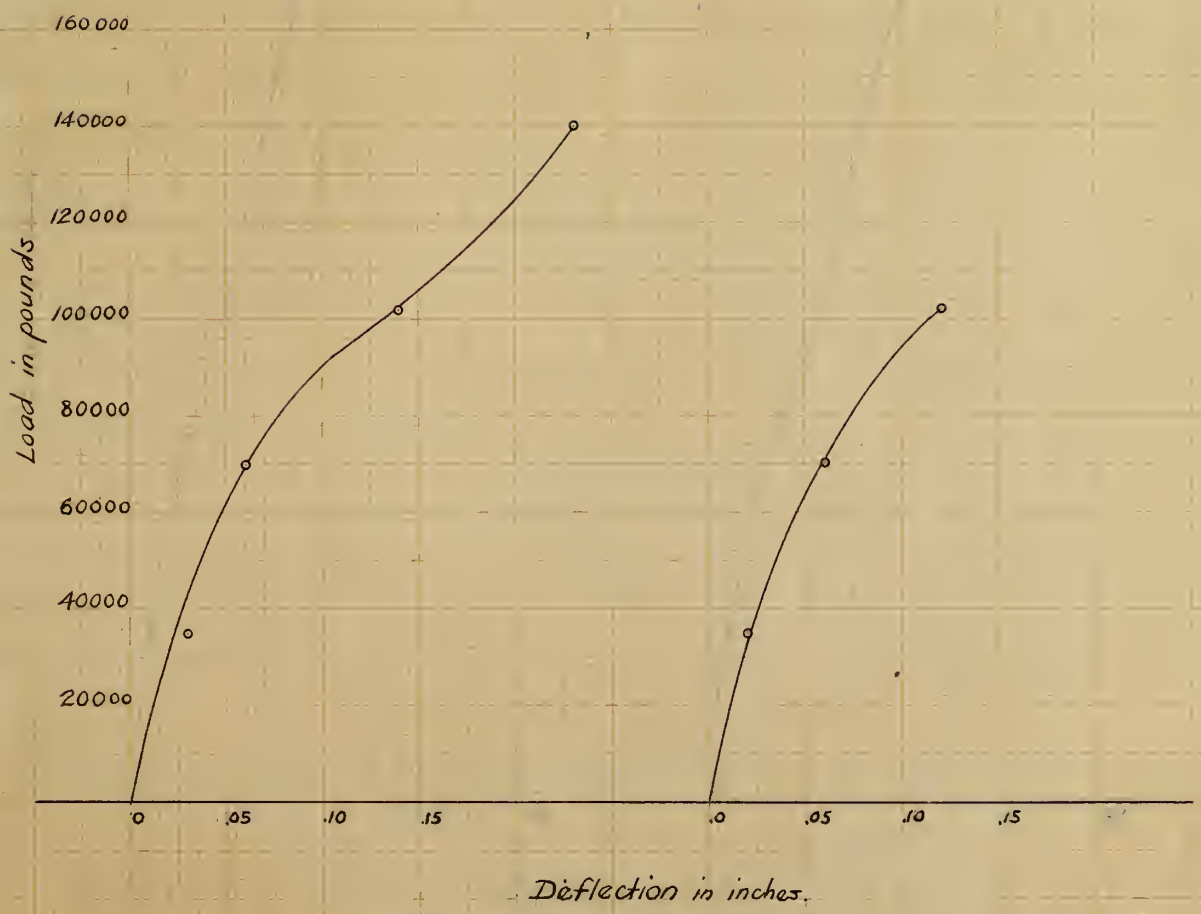

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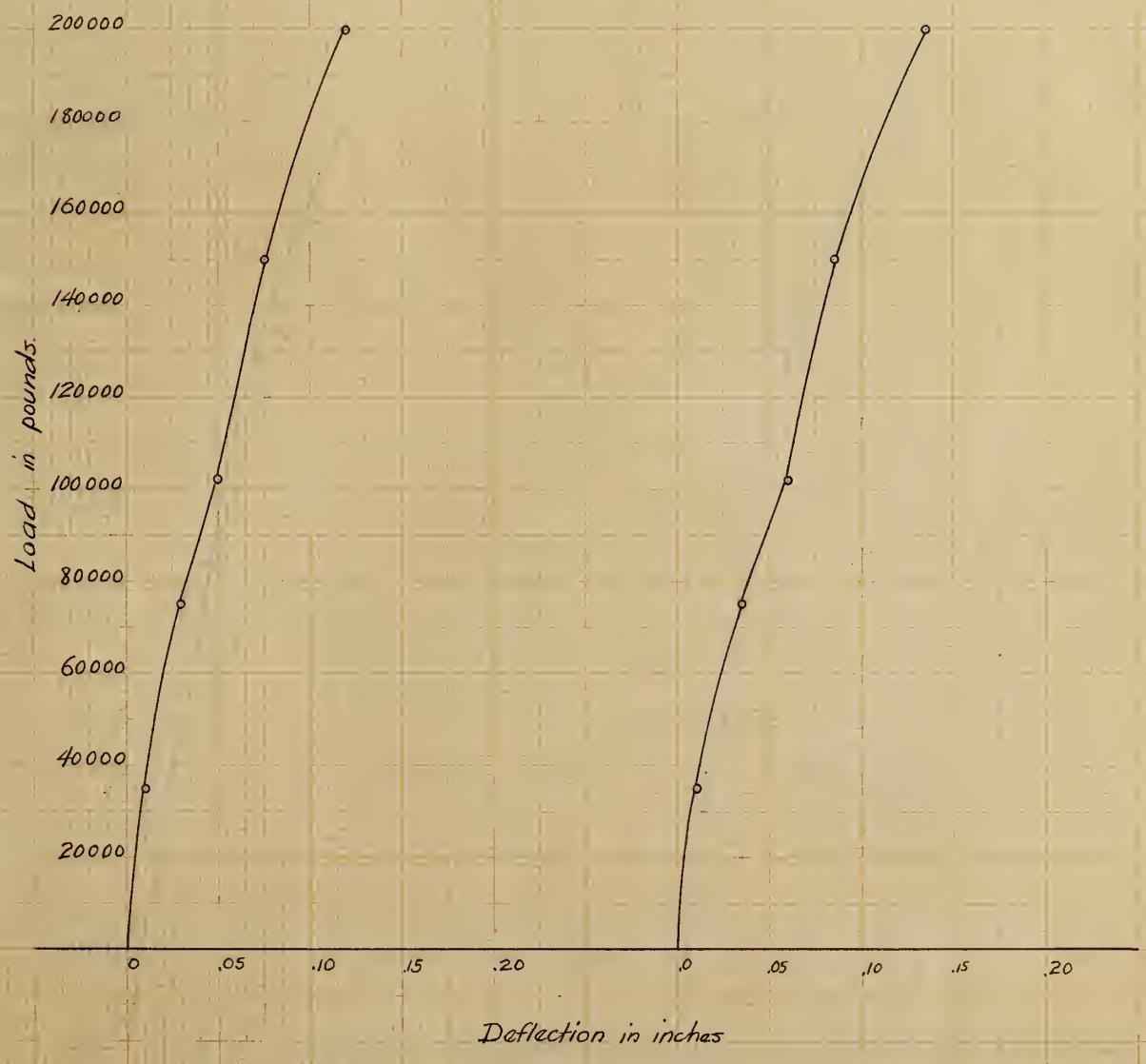

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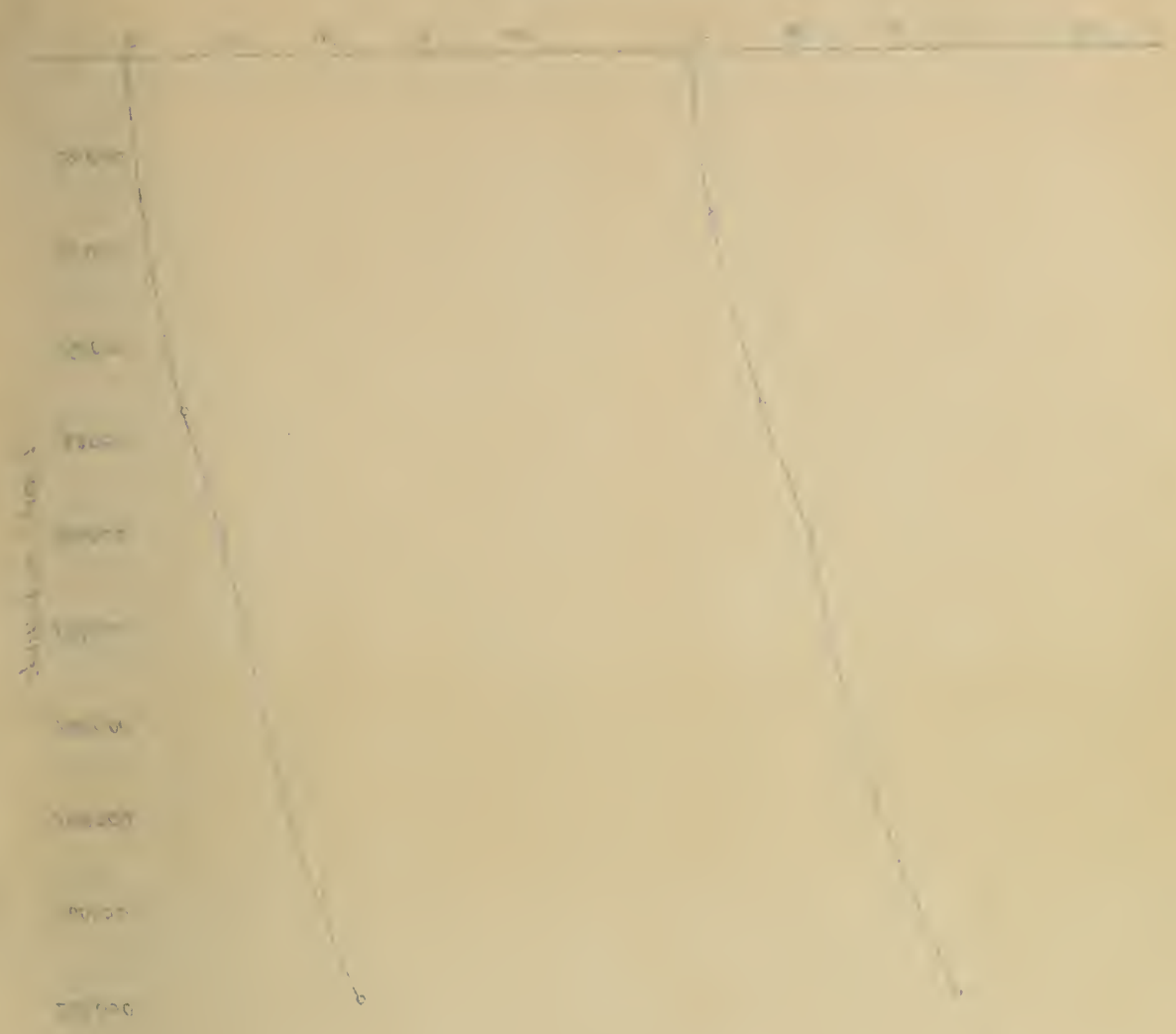

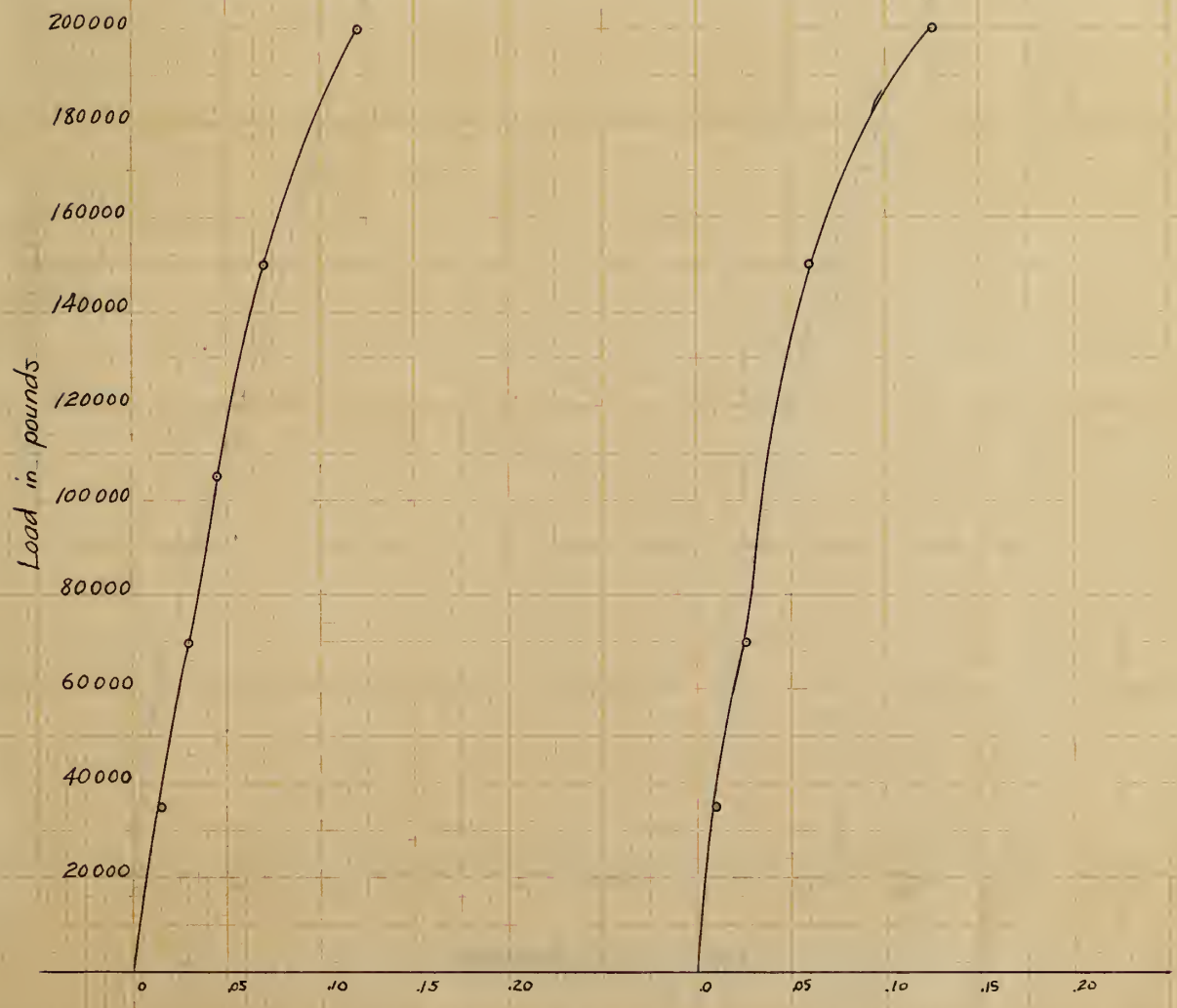

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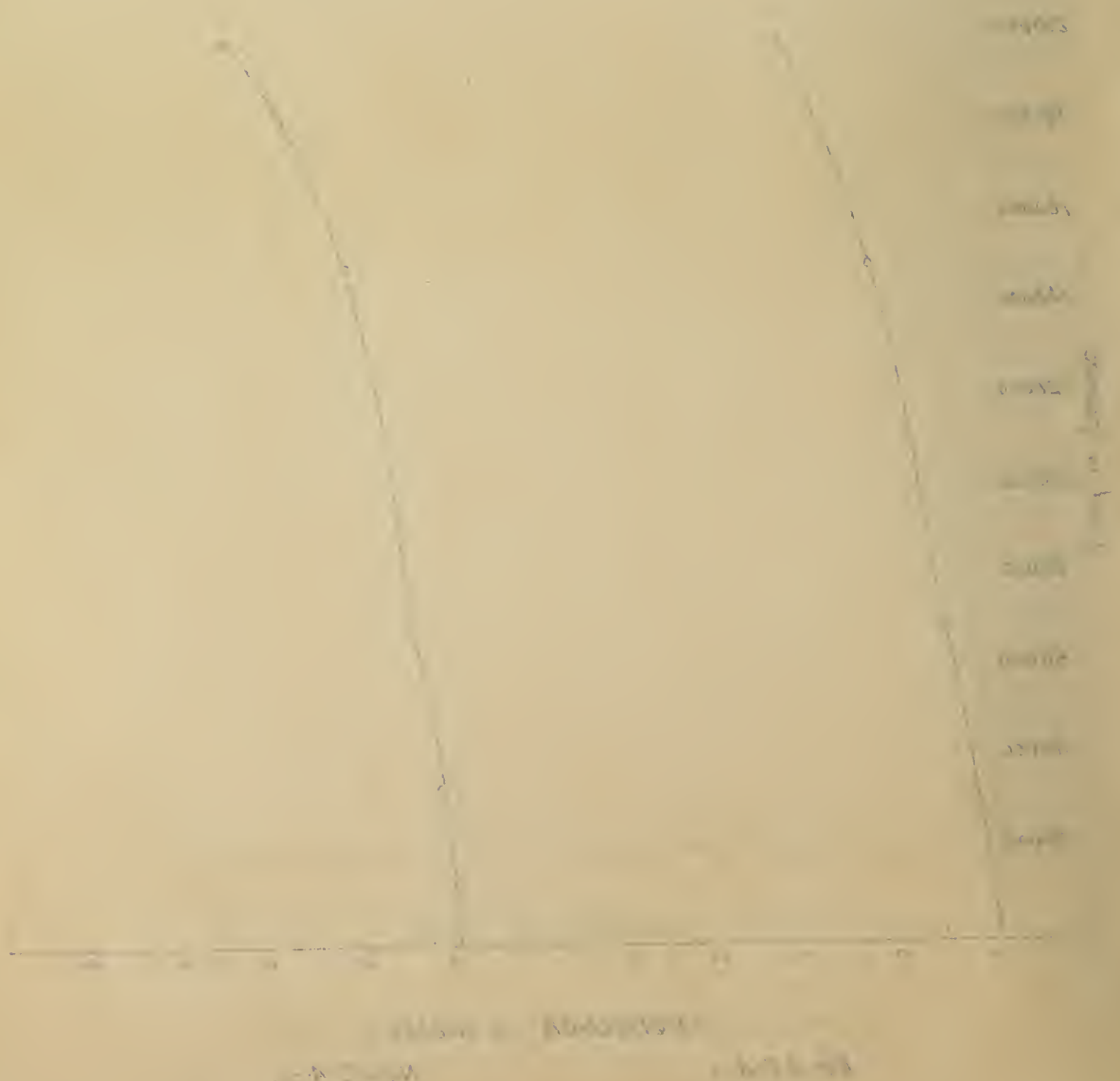

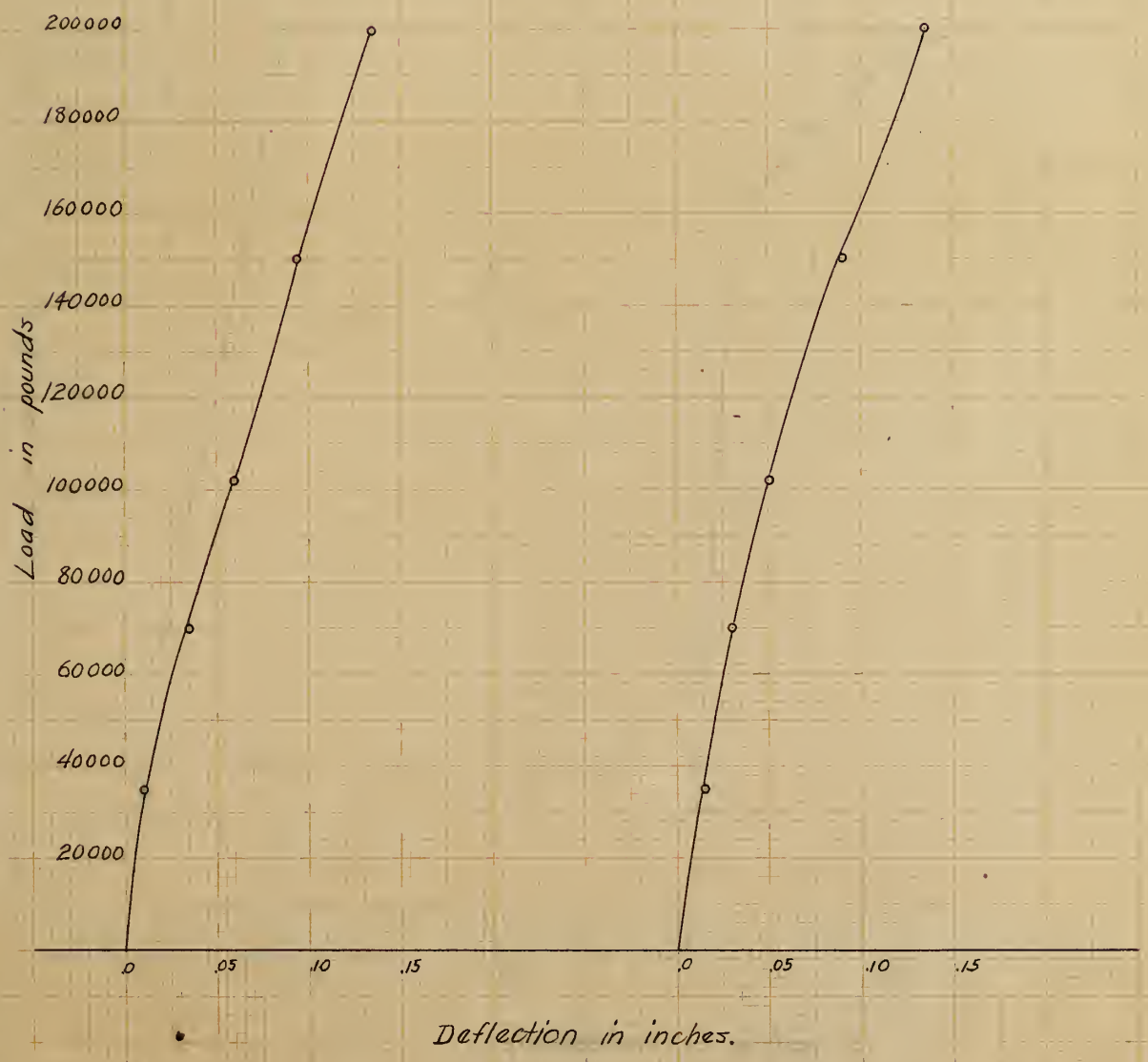

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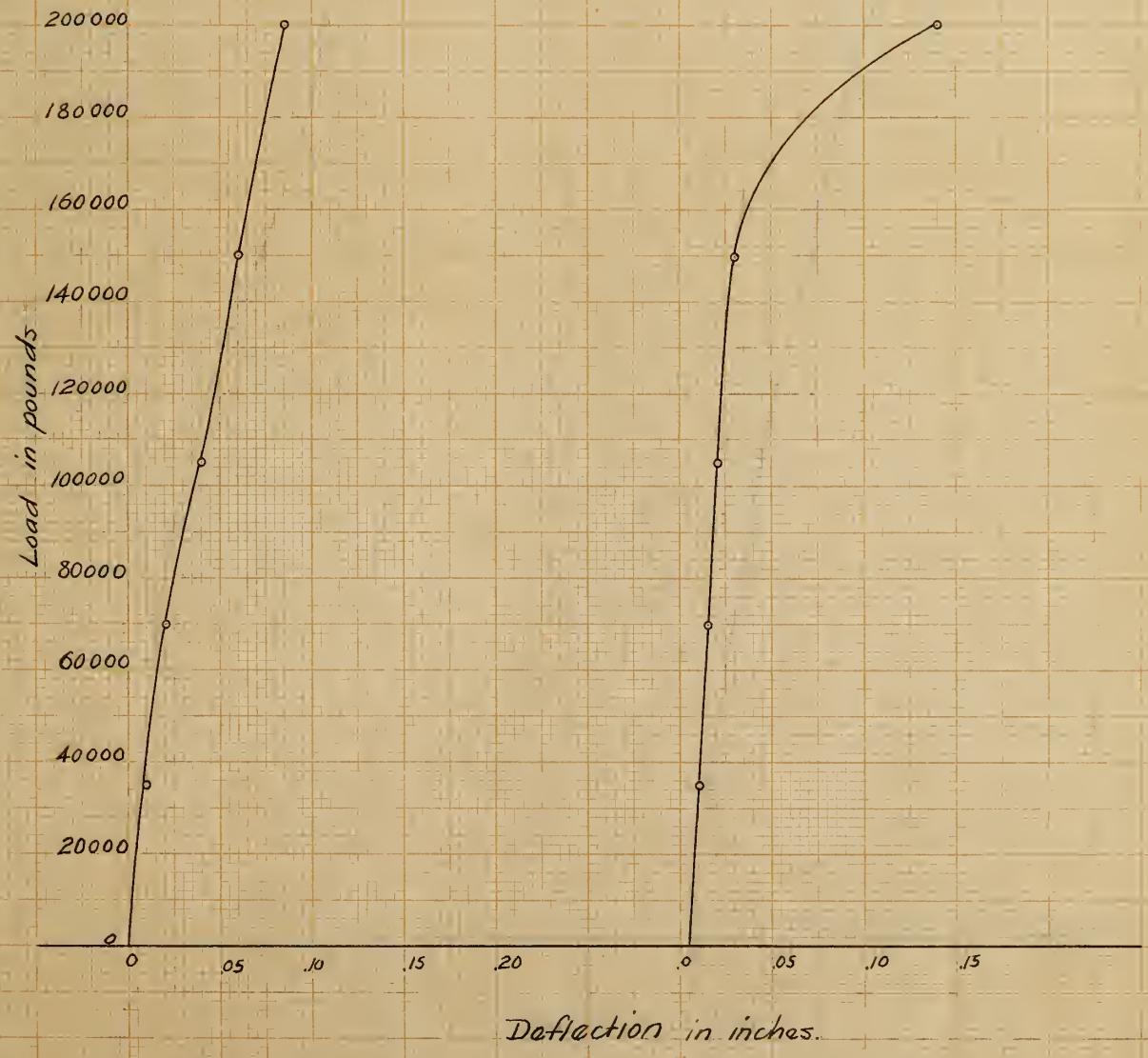

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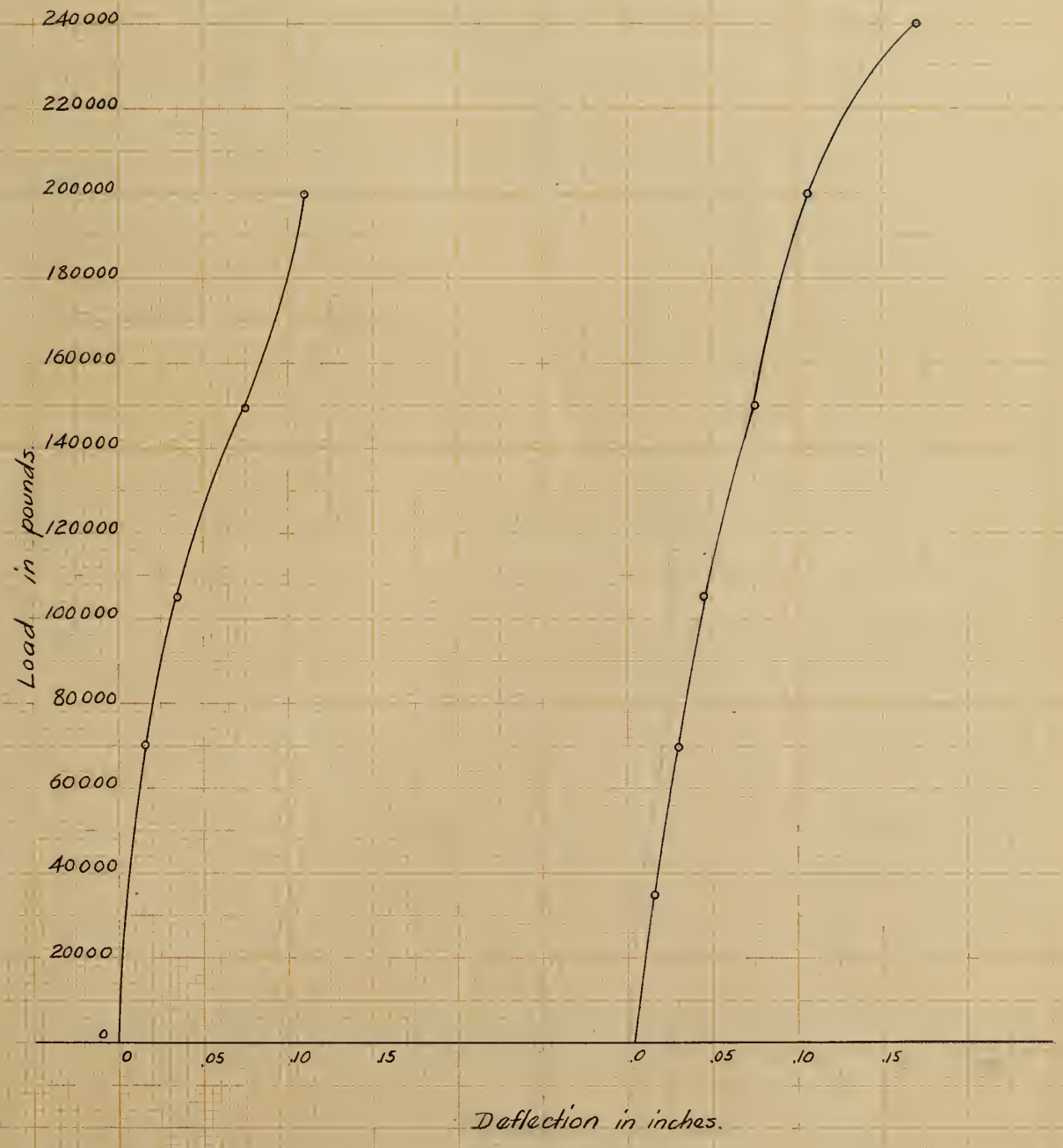
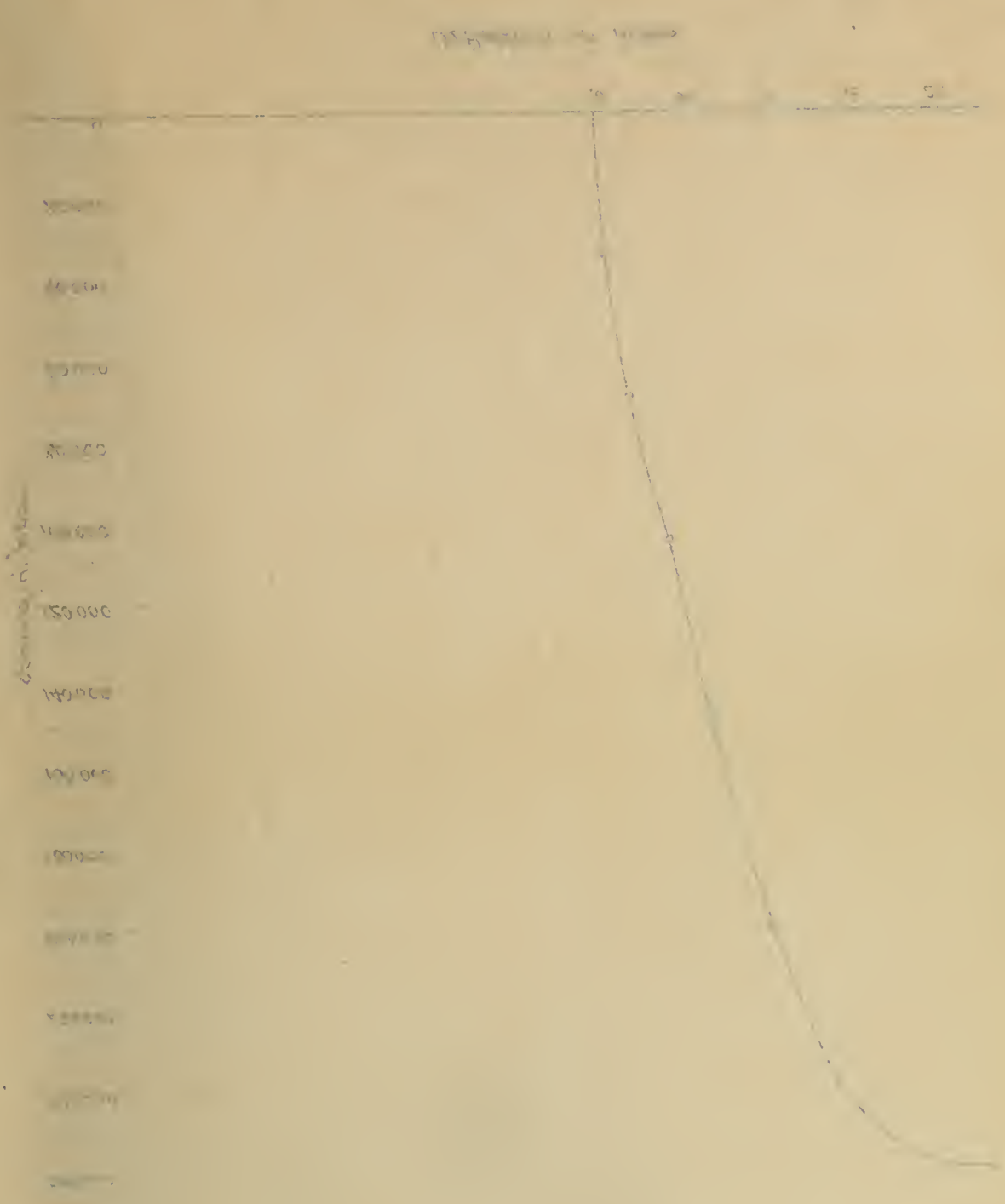


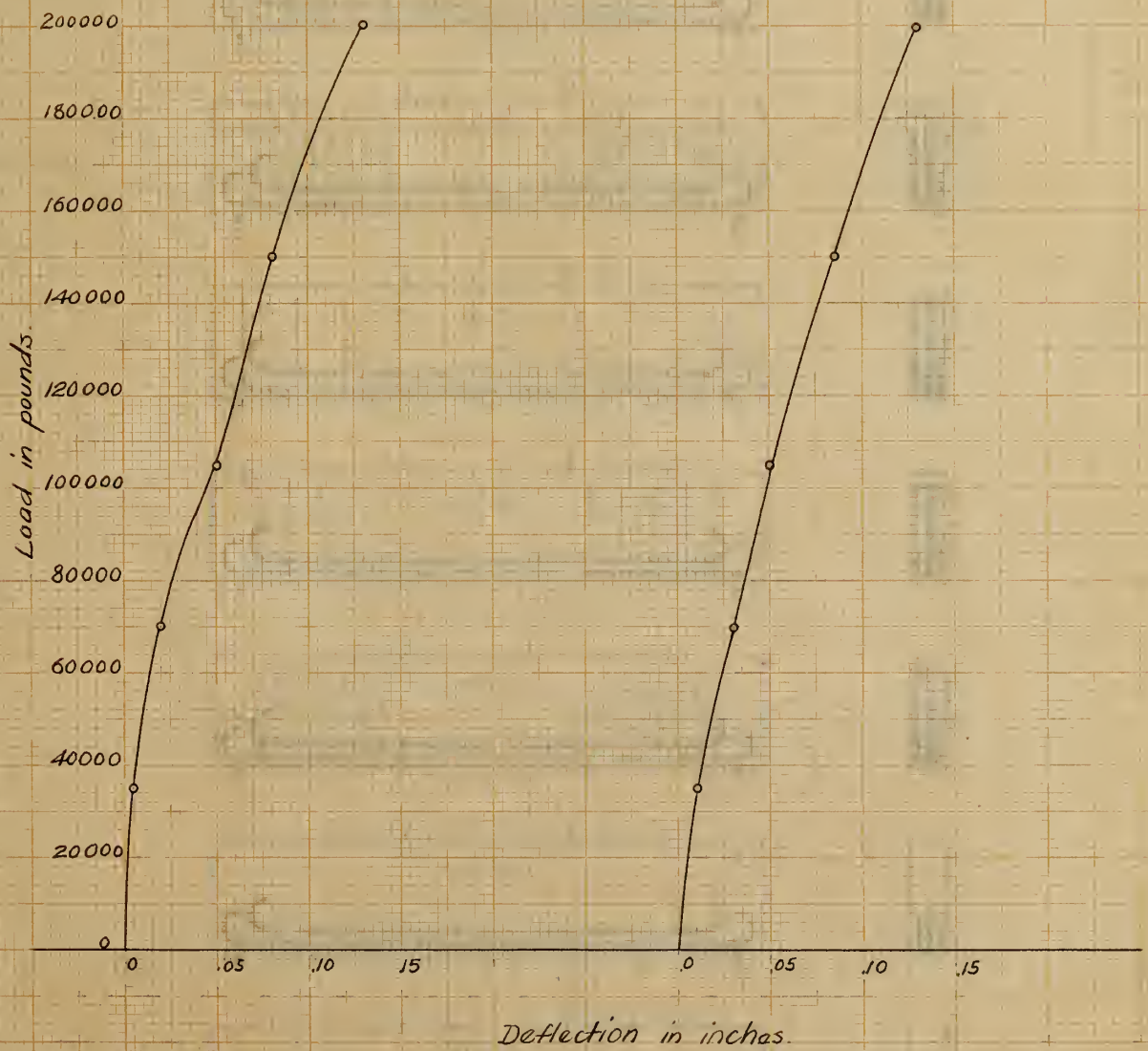
No. 22,7-1

No. 22,7-2

260000
240000
220000
200000
180000
160000
140000
120000
100000
80000
60000
40000
20000
0
Load in pounds.
.0
.05
.10
.15
.20
Deflection in inches.

No. 22,8-2





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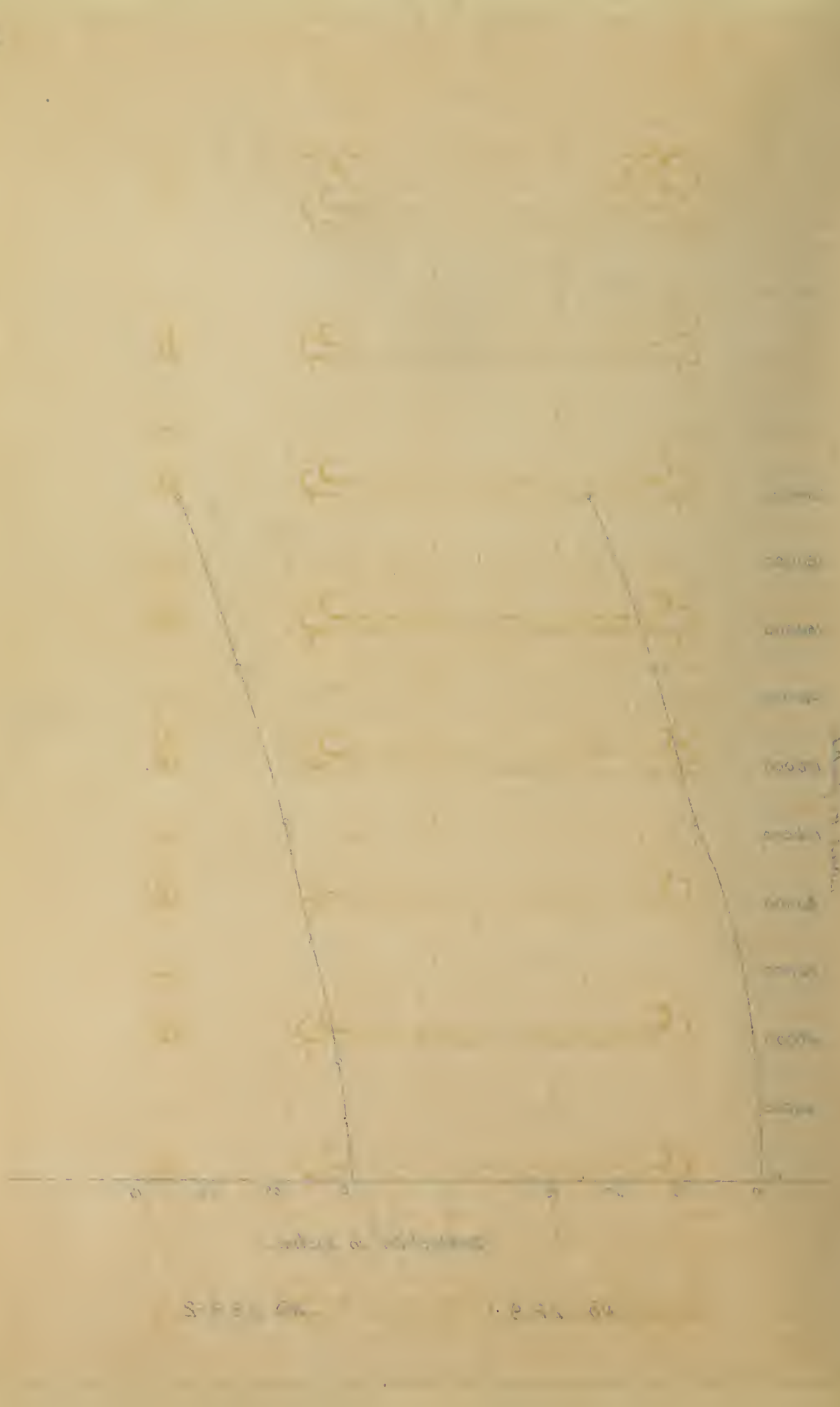
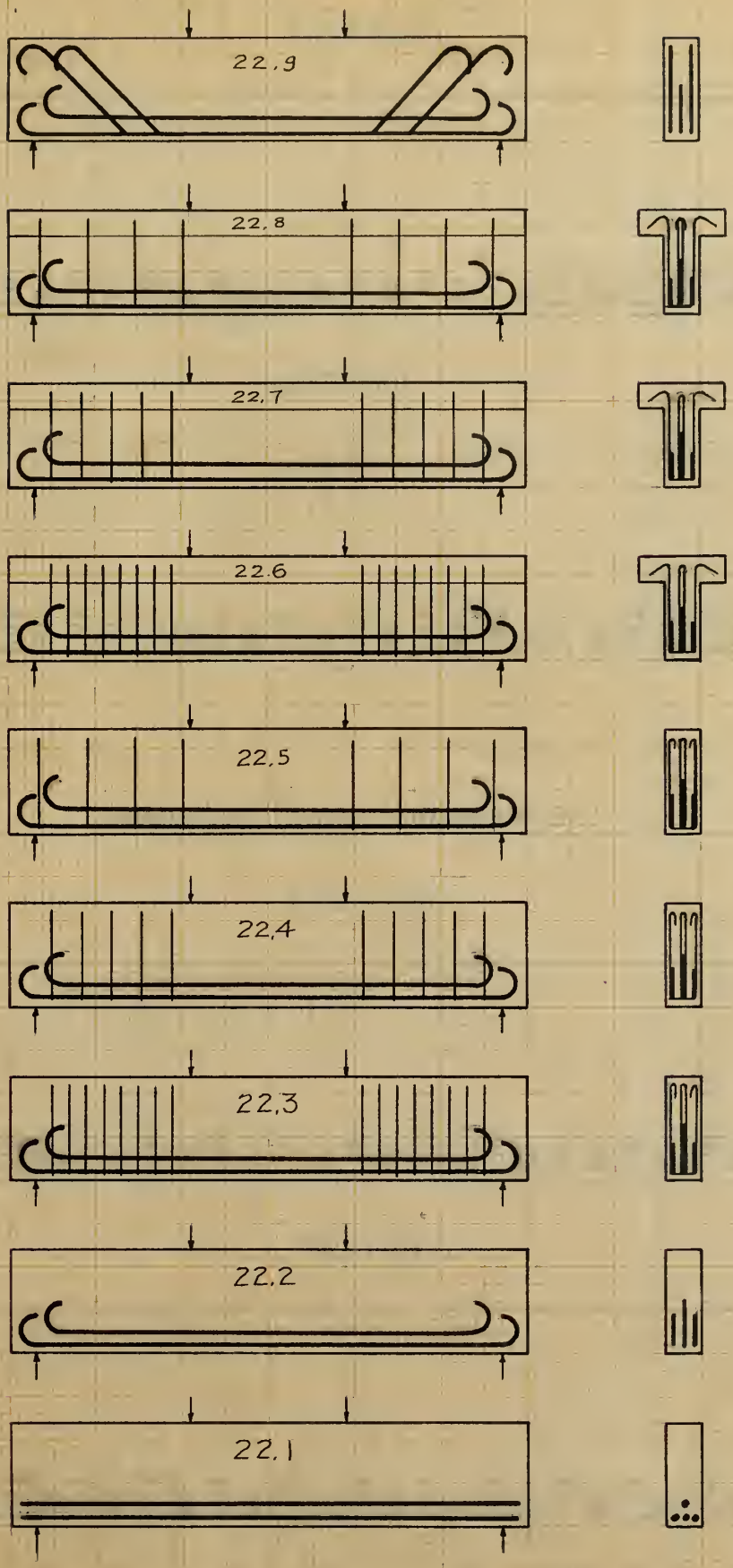


Diagrams of Test Beams



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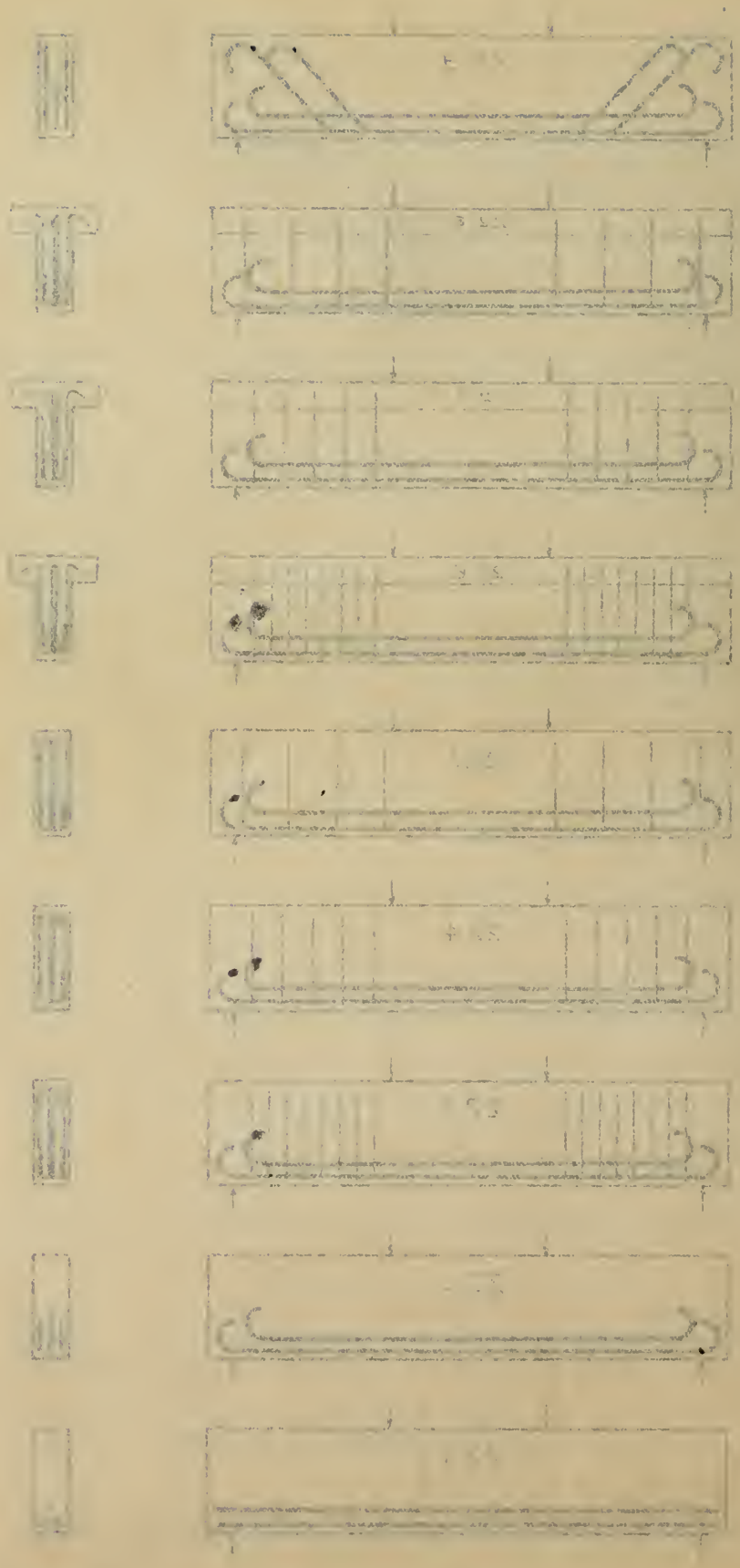

Gage Line Numbers in Beam No. 22.2

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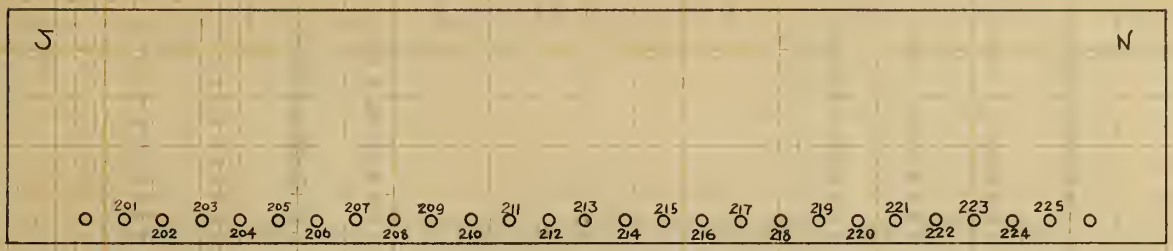
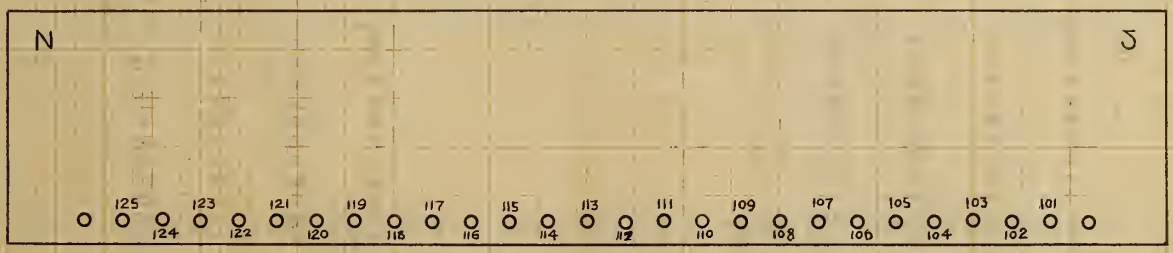
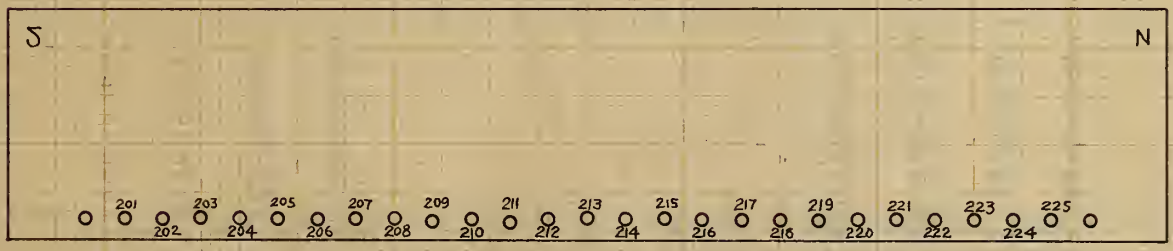


WEST SIDE.

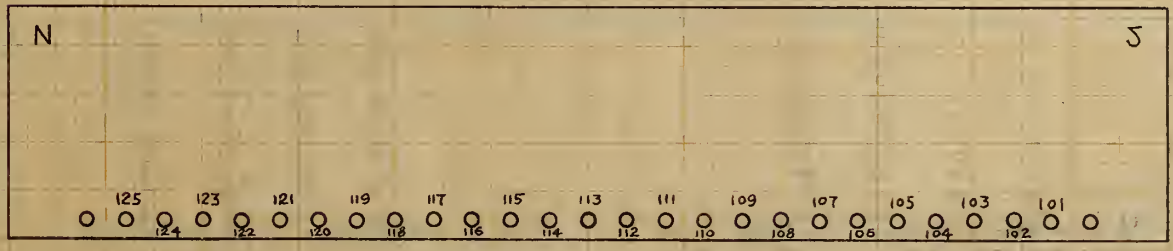


Gage Line Numbers in Beam No. 22.1

EAST SIDE.



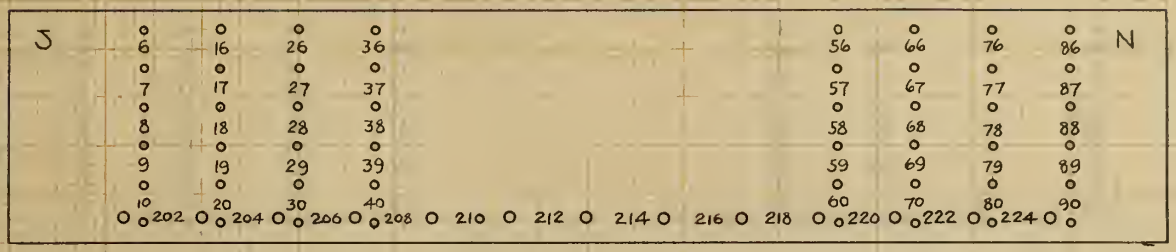
WEST SIDE.



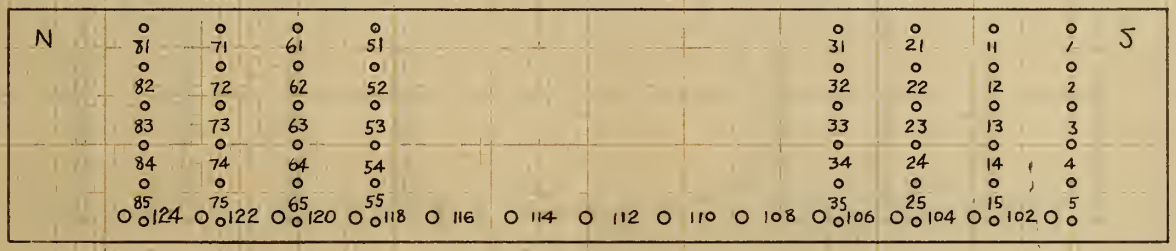
Scale 1/20

GAGE LINE NUMBERS IN BEAM NO. 22,3-2

EAST SIDE

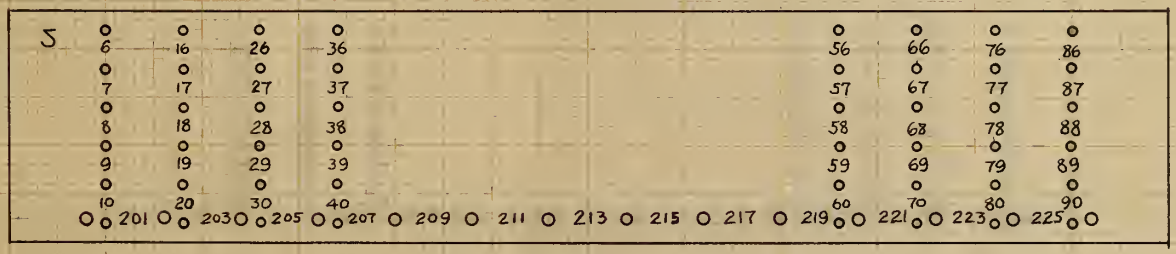


WEST SIDE



GAGE LINE NUMBERS IN BEAM NO. 22,3-1

EAST SIDE

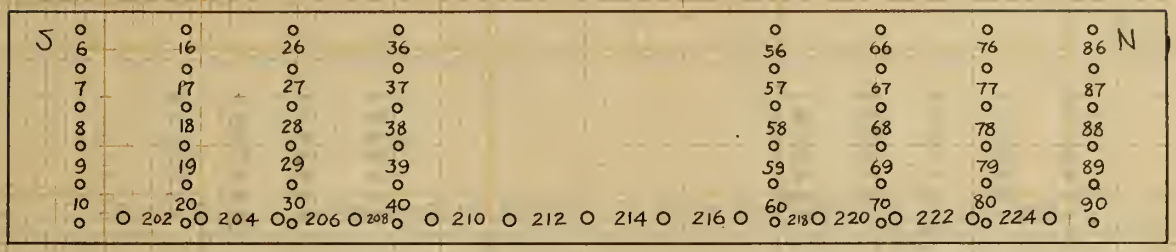


WEST SIDE

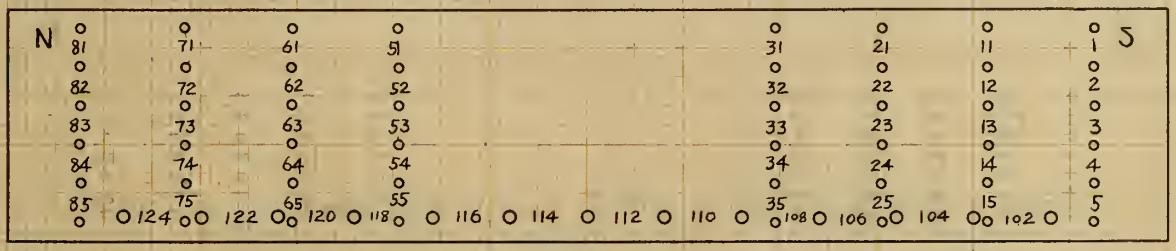
Scale 1/20

Gage Line Numbers in Beam No. 22.5

East Side

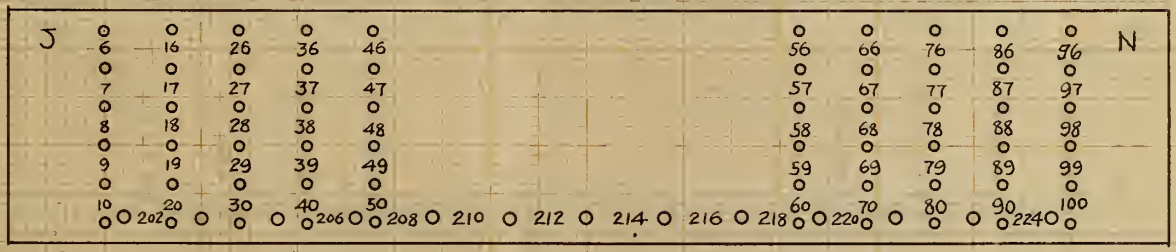


West Side

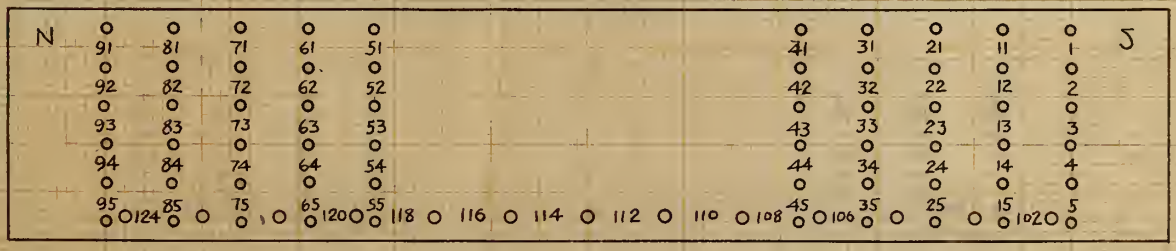


Gage Line Numbers in Beam No. 22.4

East Side



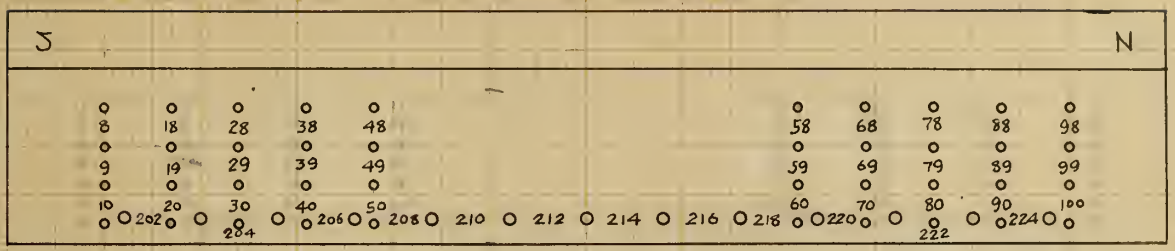
West Side



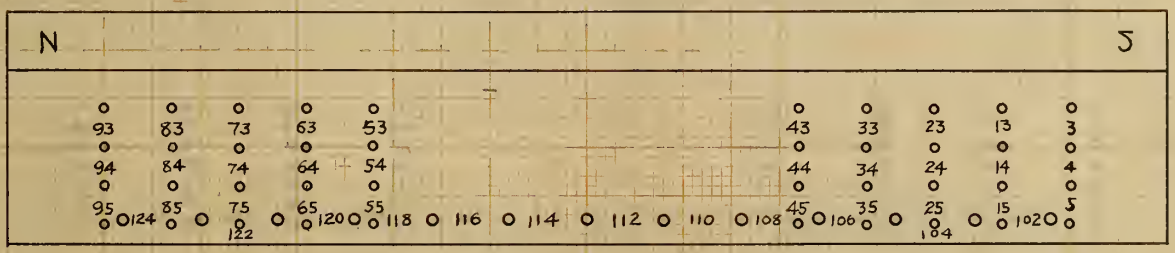
Scale 1/20

Gage Line Numbers in Beam No. 22.7

EAST SIDE

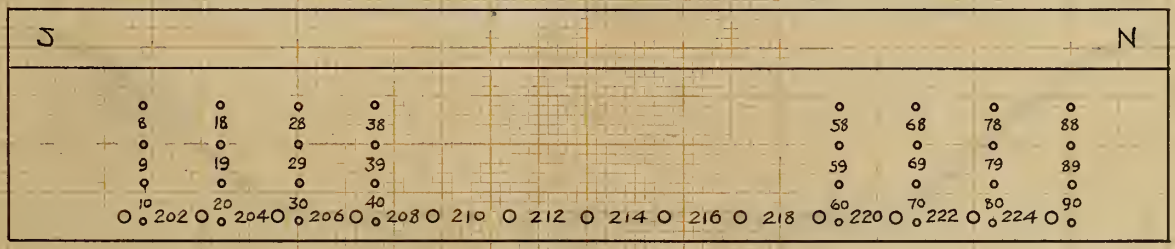


WEST SIDE

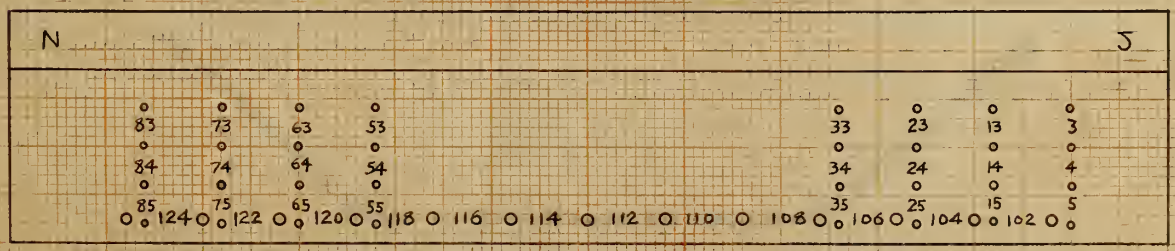


Gage Line Numbers in Beam No. 22.6.

EAST SIDE



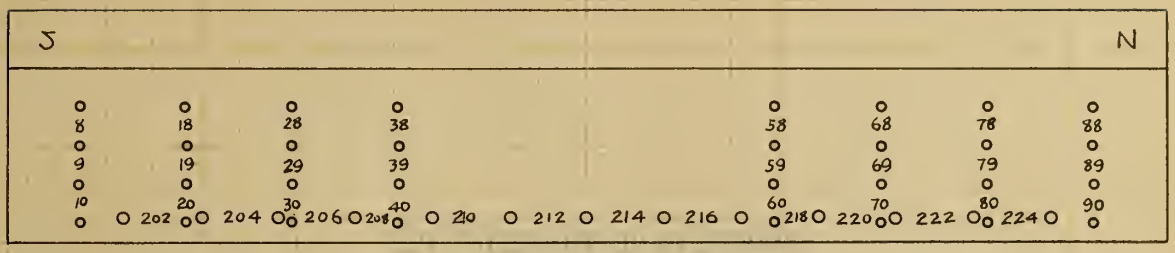
WEST SIDE



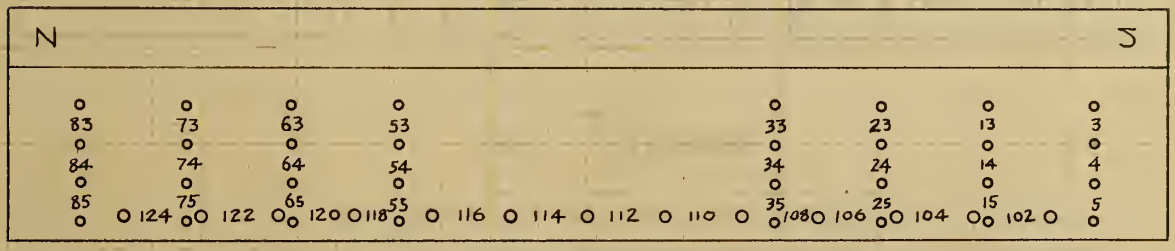
Scale 1/20

GAGE LINE NUMBERS IN BEAM NO. 22,8

EAST SIDE

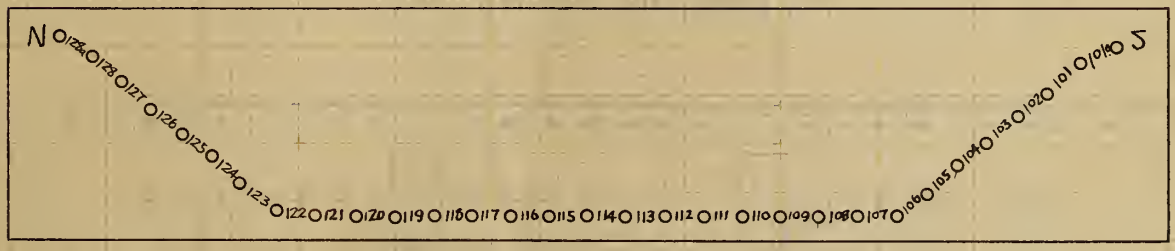


WEST SIDE

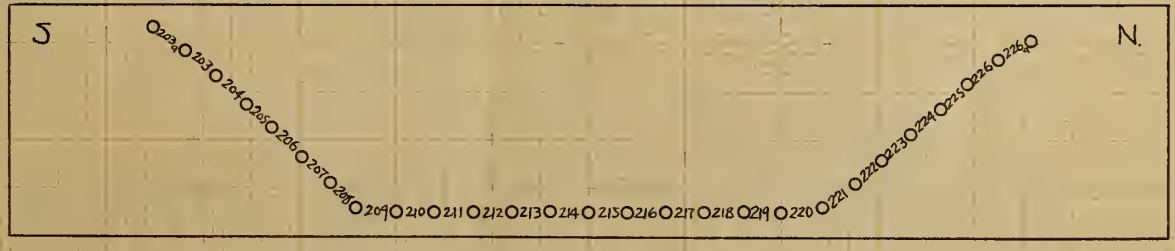


GAGE LINE NUMBERS IN BEAM NO. 22.9

WEST SIDE



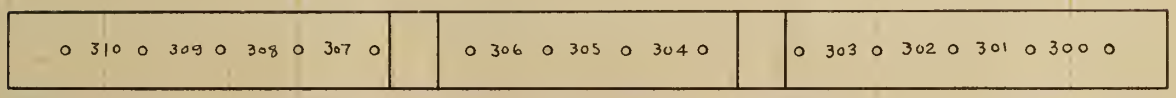
EAST SIDE.



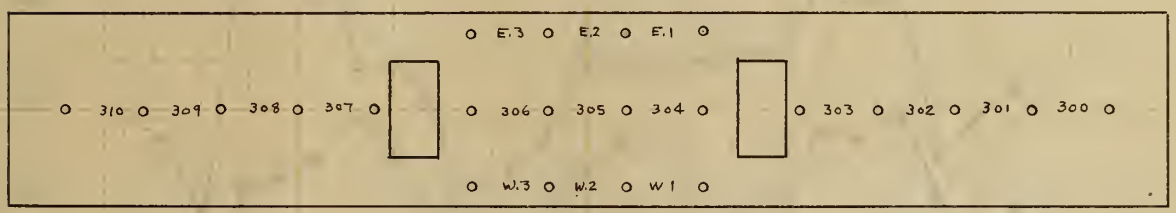
Scale 1/20

Gage Line Numbers at The Top Surface of The Beams

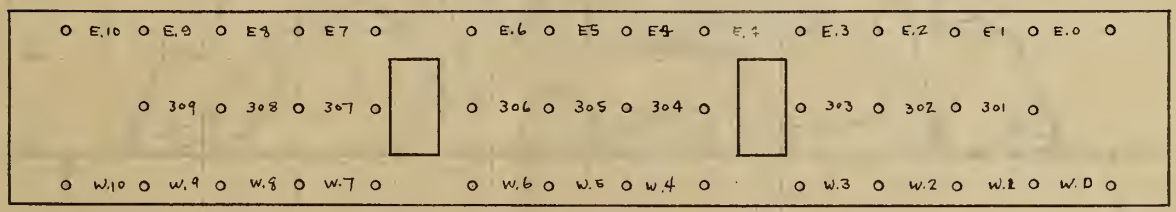
In Rectangular Beams

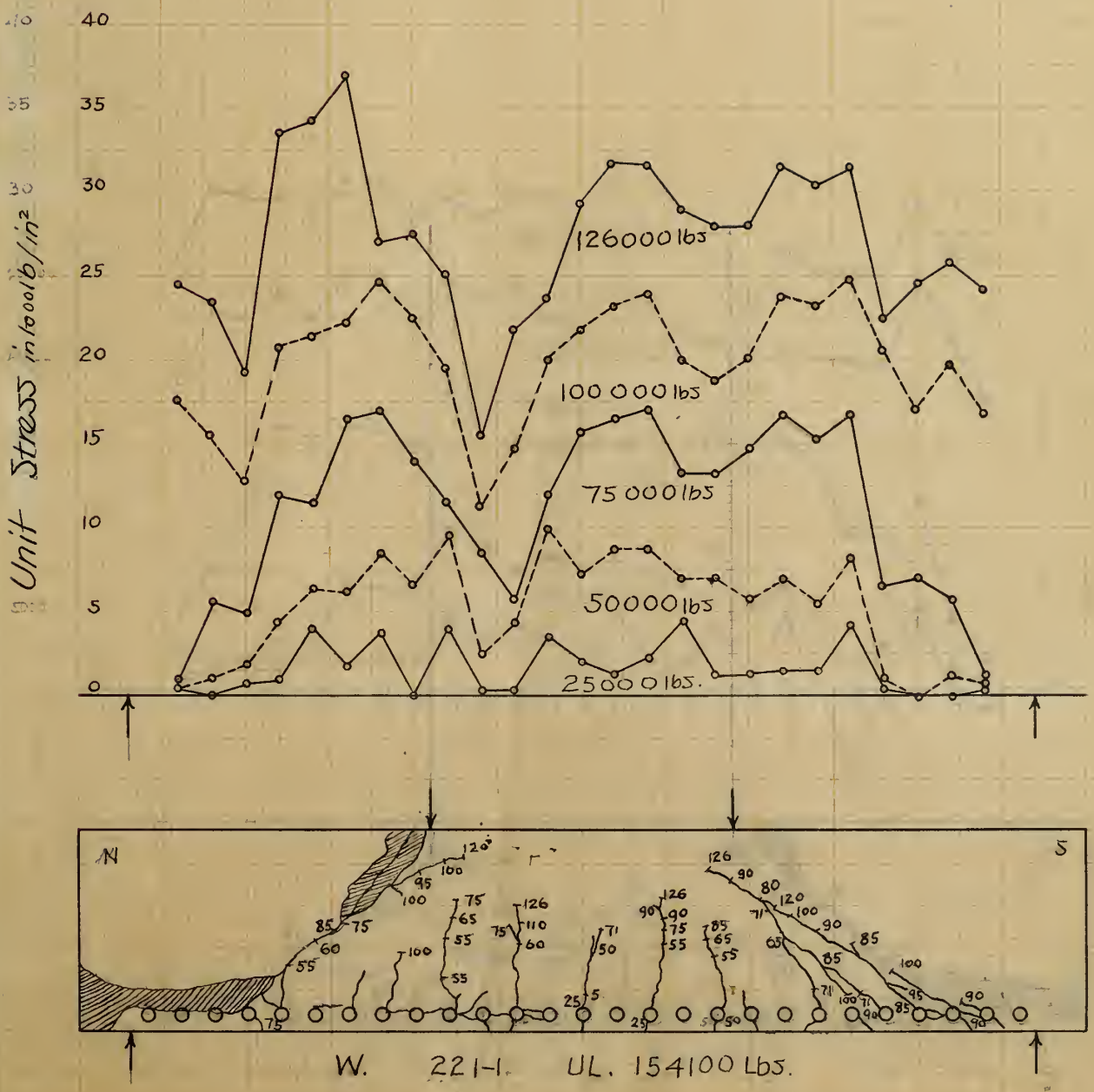


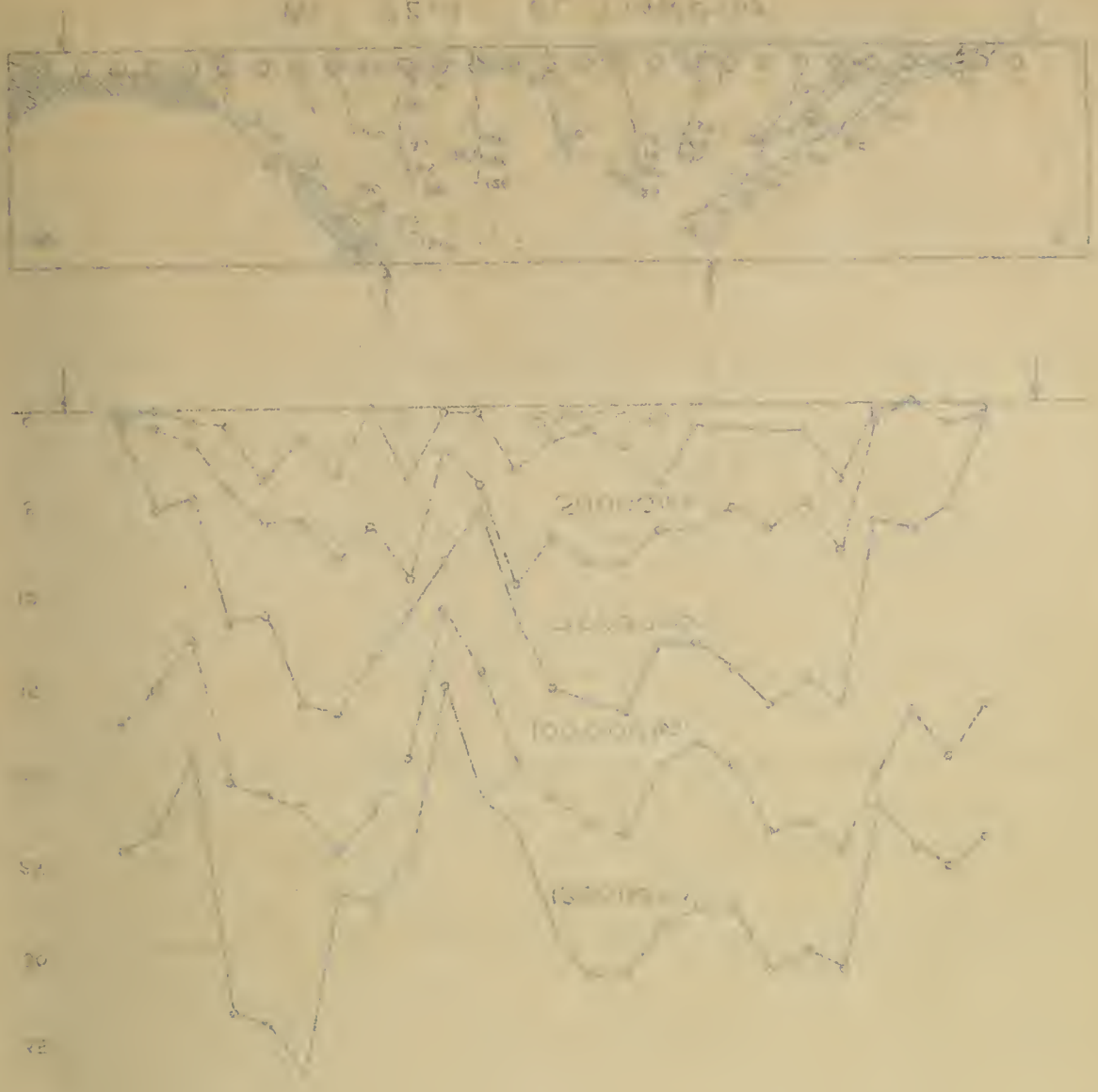
In T- Beams

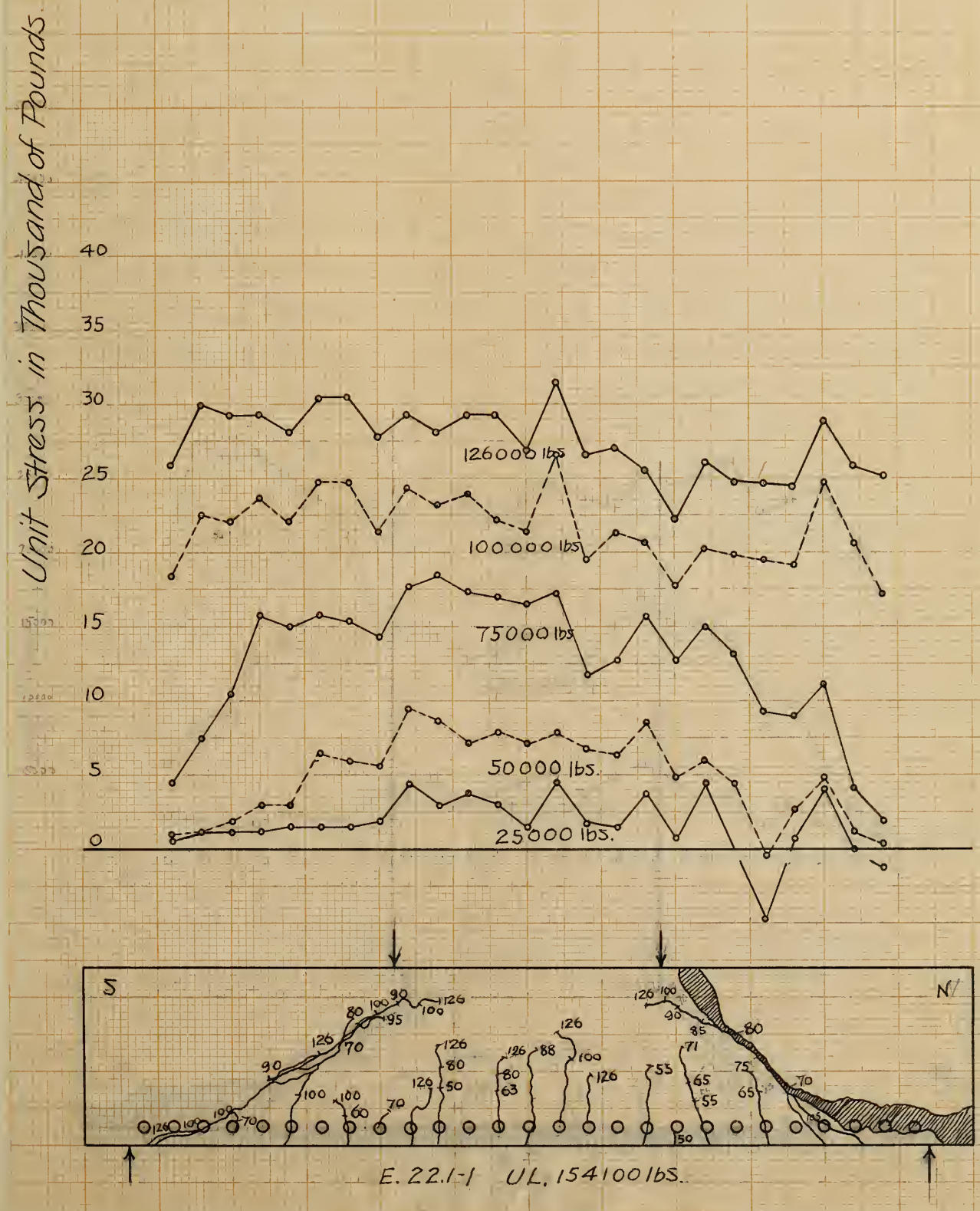


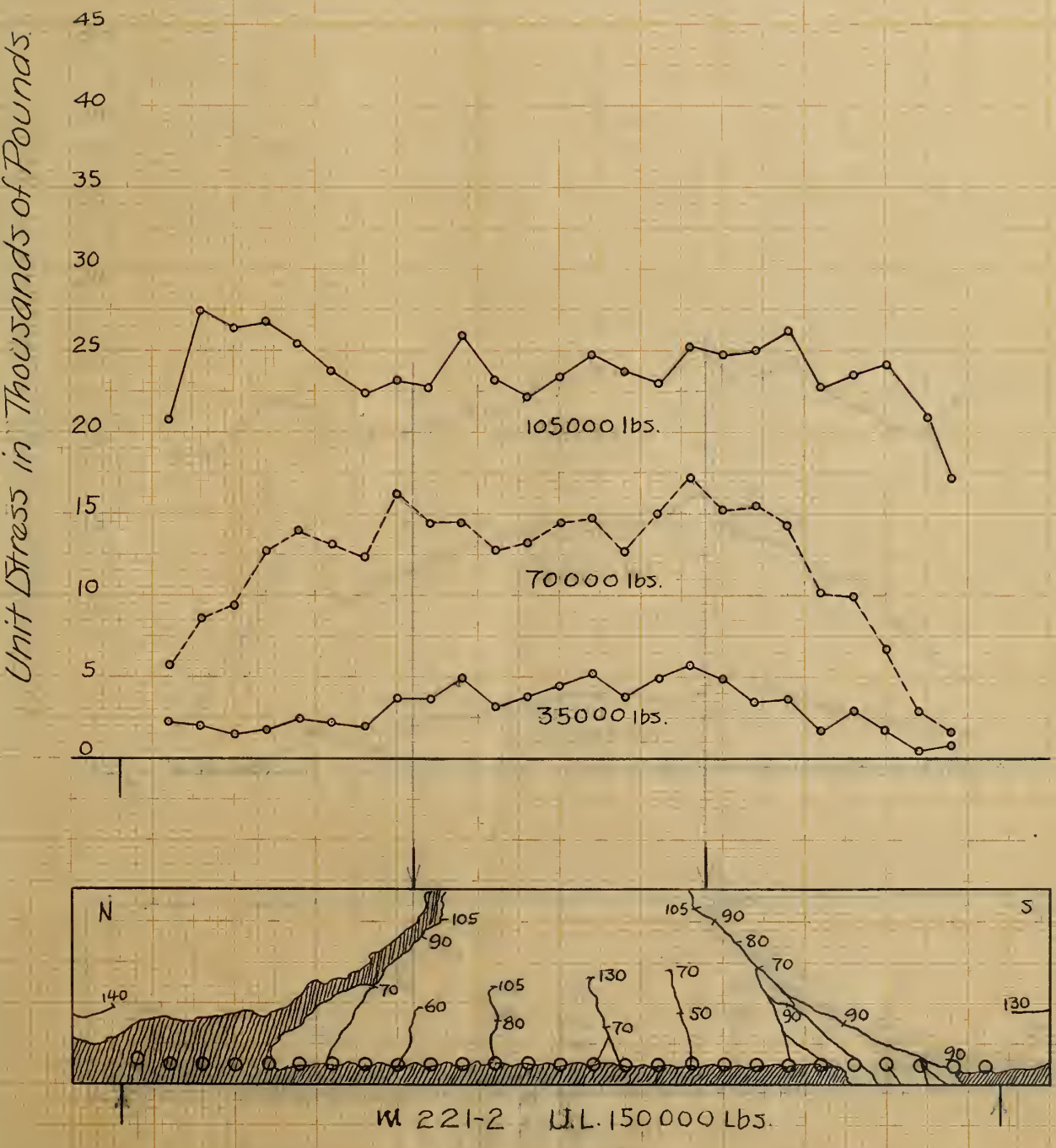
In T-Beam No. 226-2.



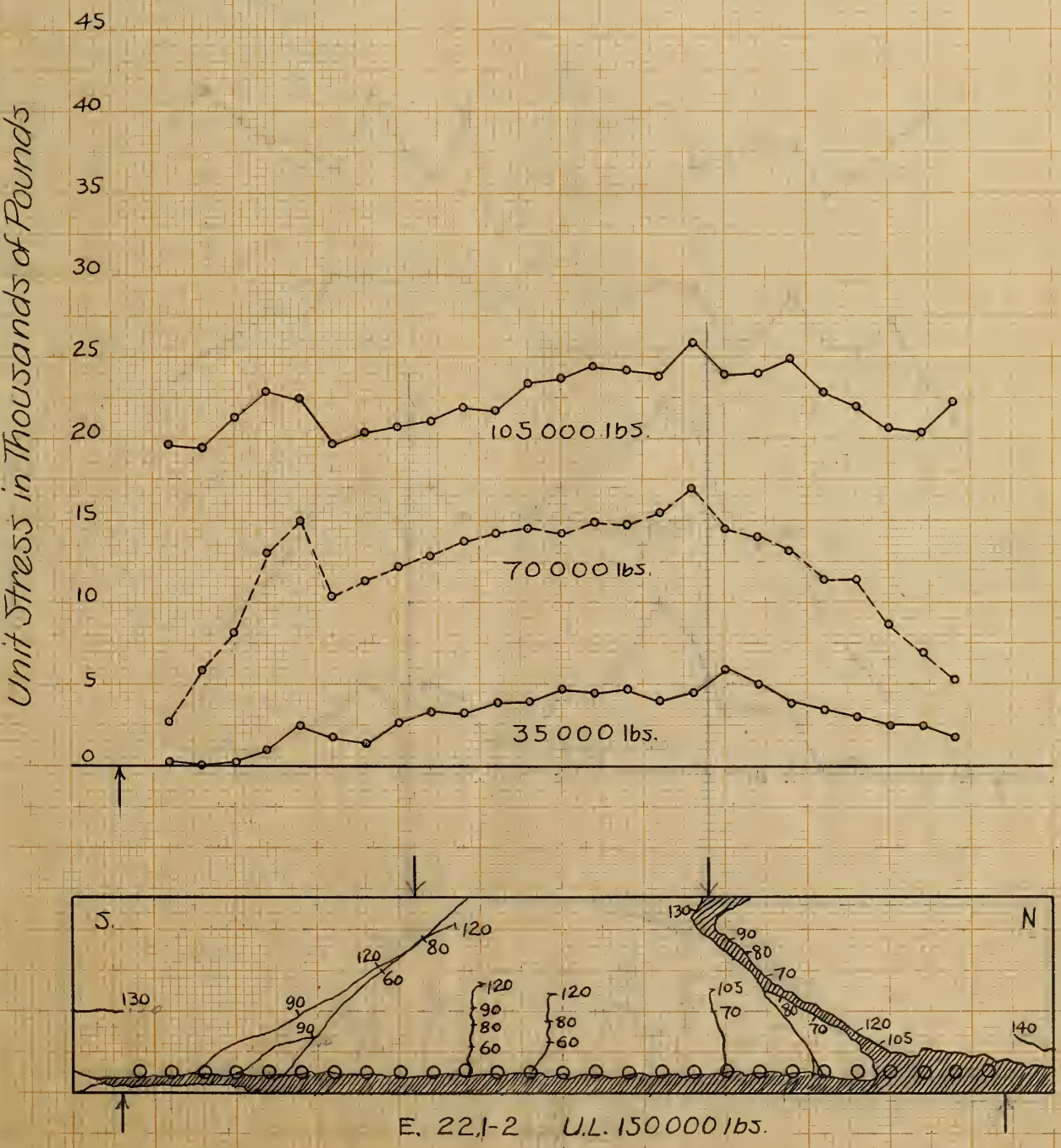

Unit Stress in lbs/in²
40
35
30
25
20
15
10
5
0
126000 lbs
100 000 lbs
75 000 lbs
50000 lbs
25000 lbs.
N
S
W. 221-1. UL. 154100 Lbs.



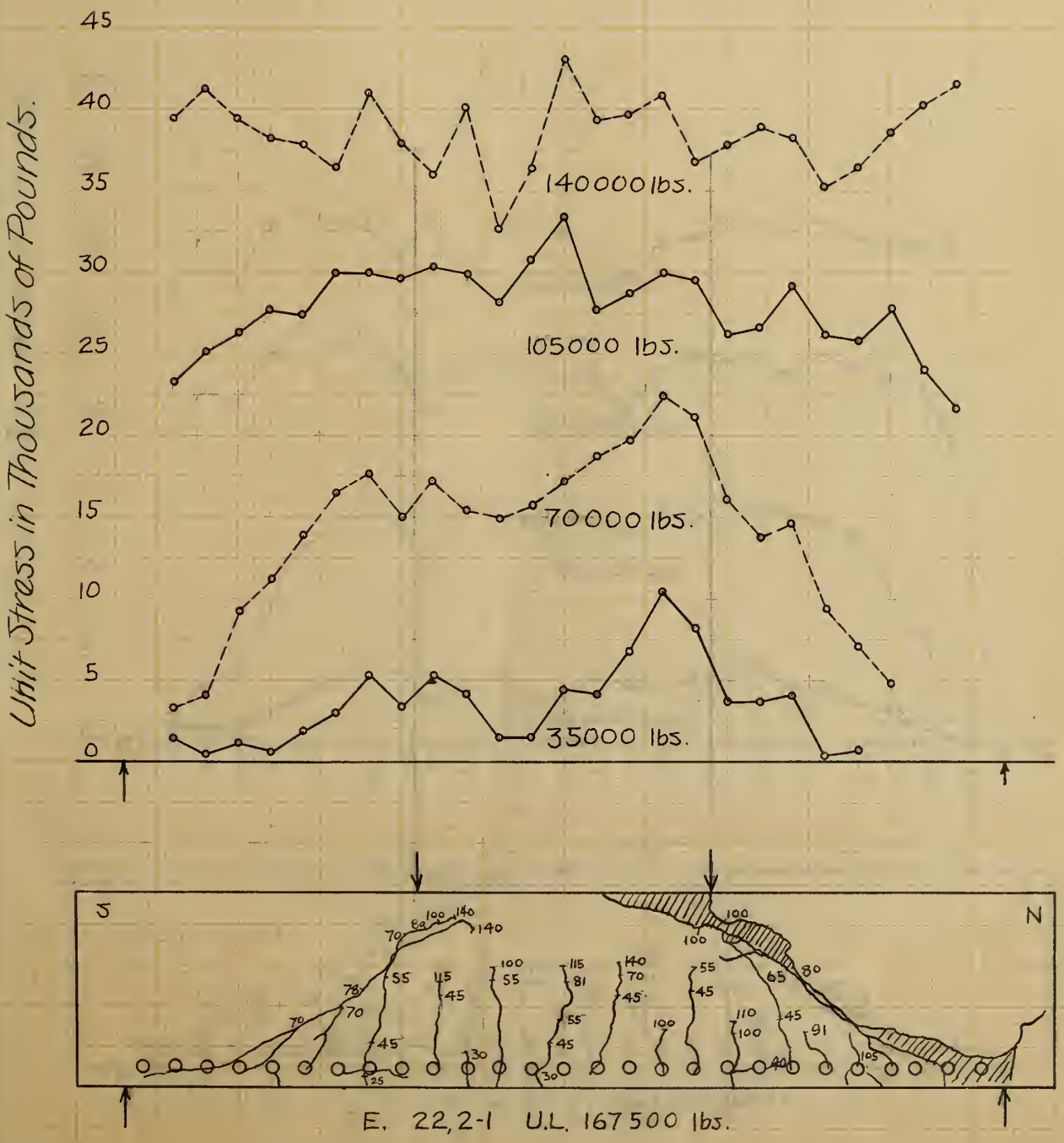

Unit Stress in Thousand of Pounds.
40
35
30
25
20
15
10
5
0
126000 lbs.
100000 lbs.
75000 lbs.
50000 lbs.
25000 lbs.
S
N
E. 22.1-1 UL. 154100 lbs.

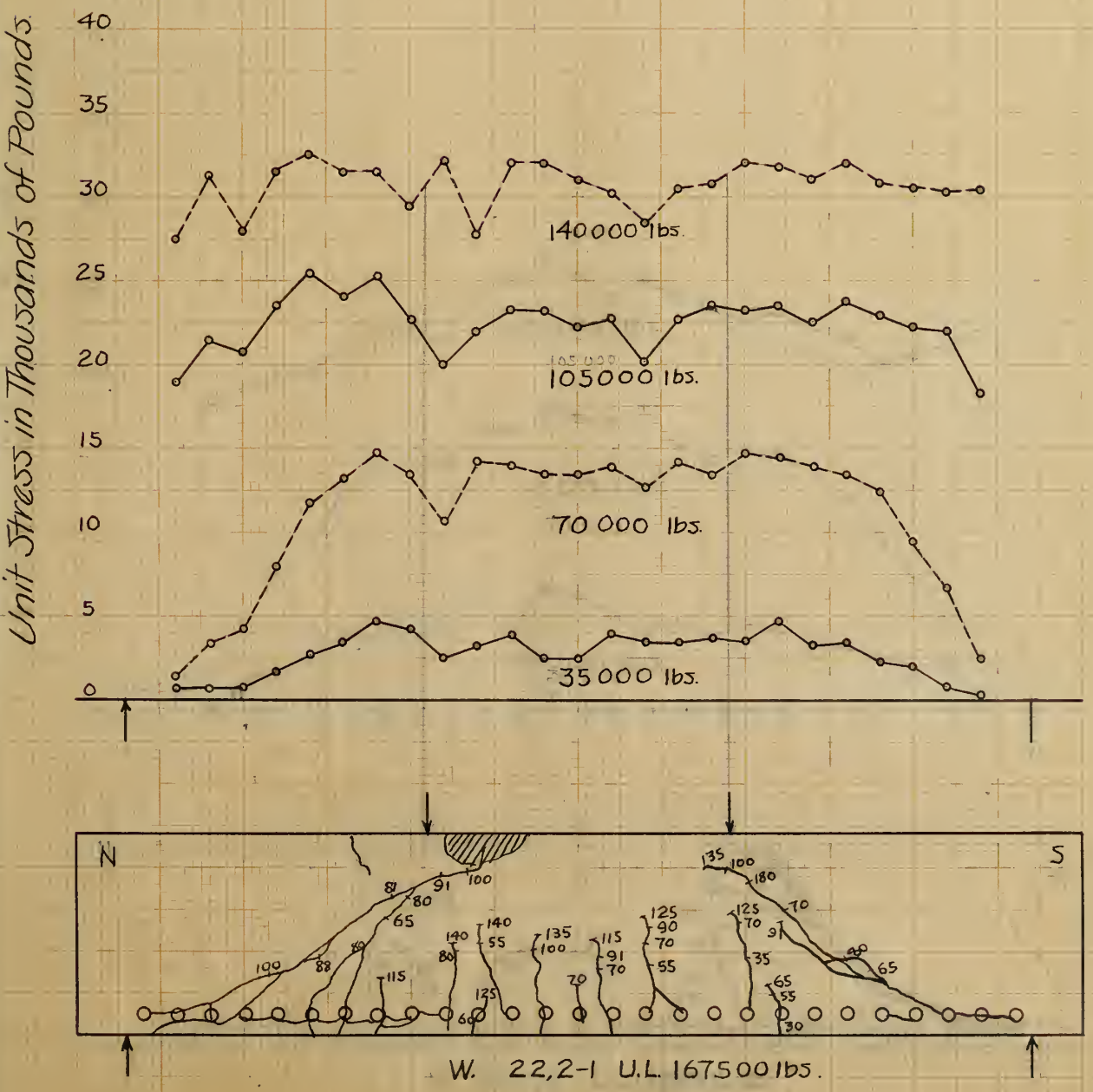


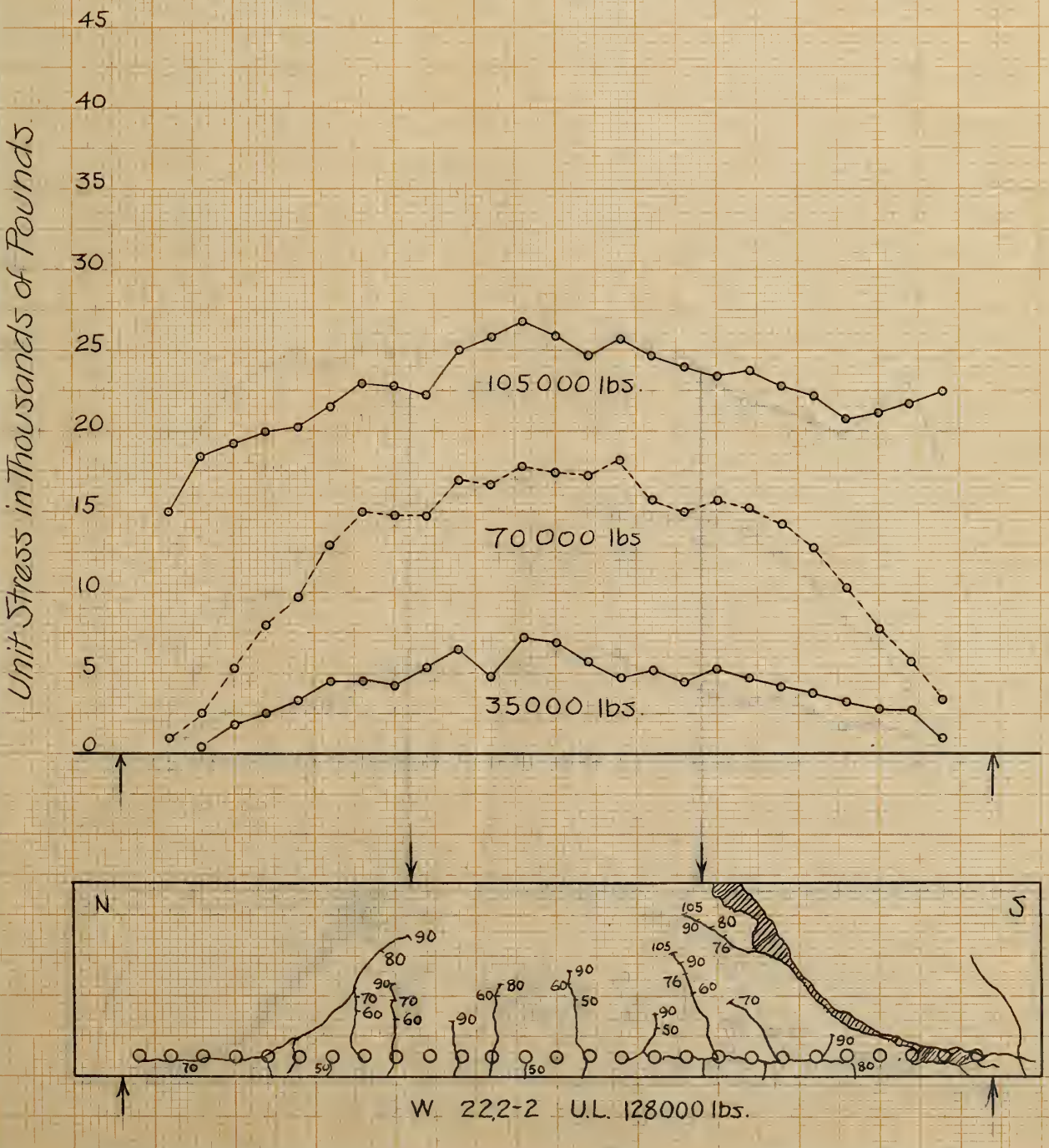
M 221-2 U.L. 150000 Lbs.



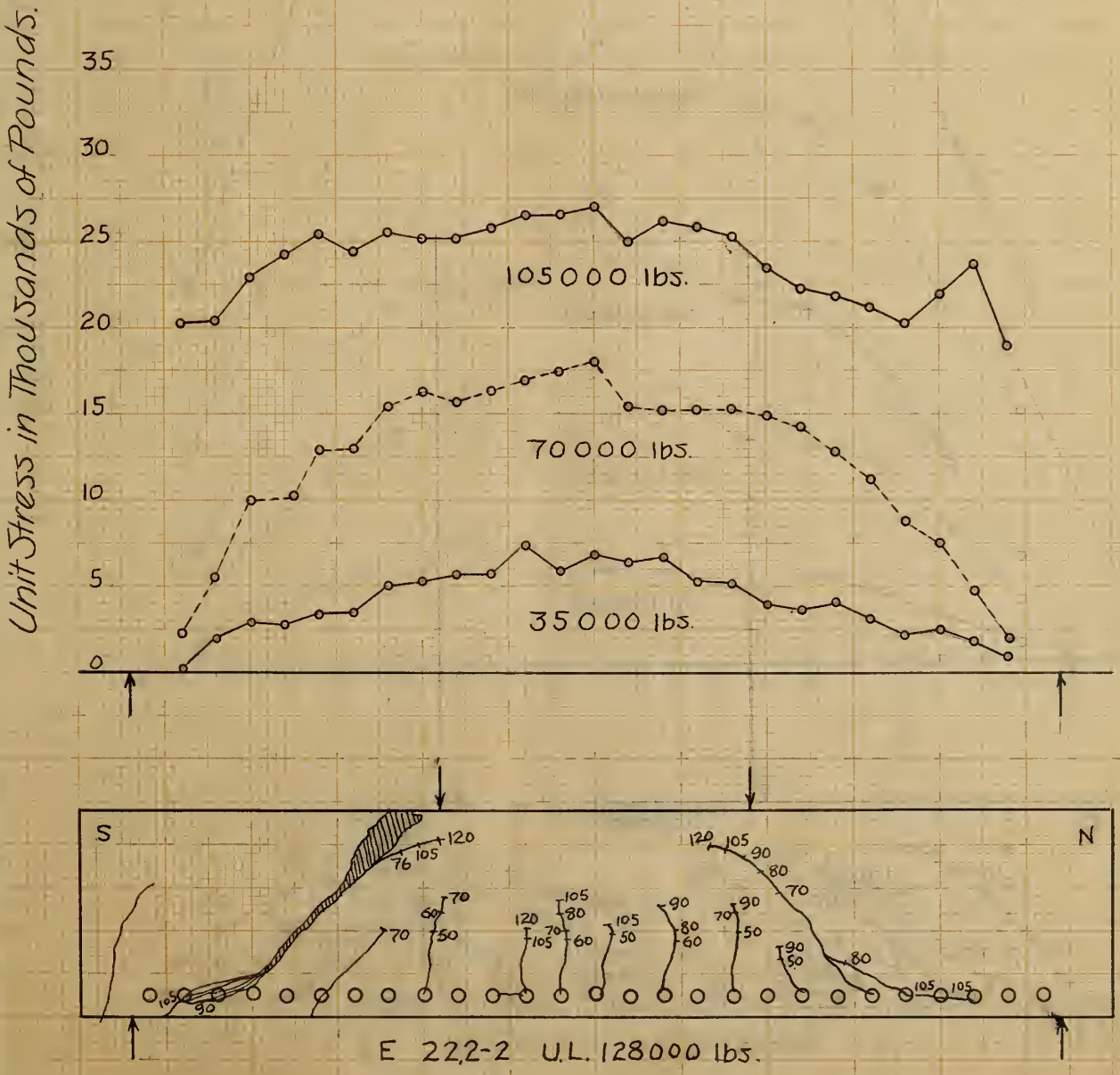
E. 22,1-2 U.L. 150000 lbs.

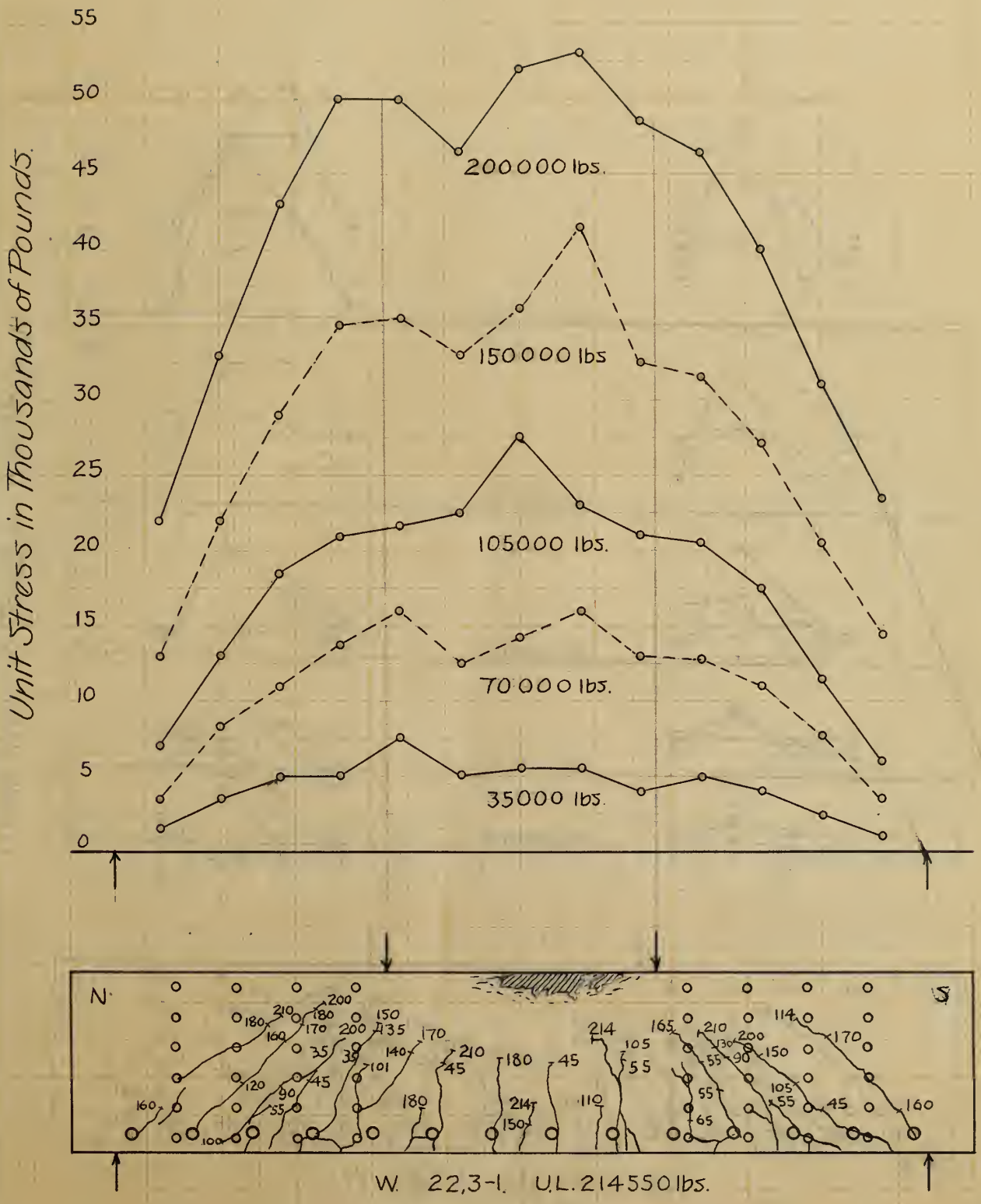

Unit Stress in Thousands of Pounds.
45
40
35
30
25
20
15
10
5
0
140000 lbs.
105000 lbs.
70000 lbs.
35000 lbs.
S
N
E. 22,2-1 U.L. 167500 lbs.

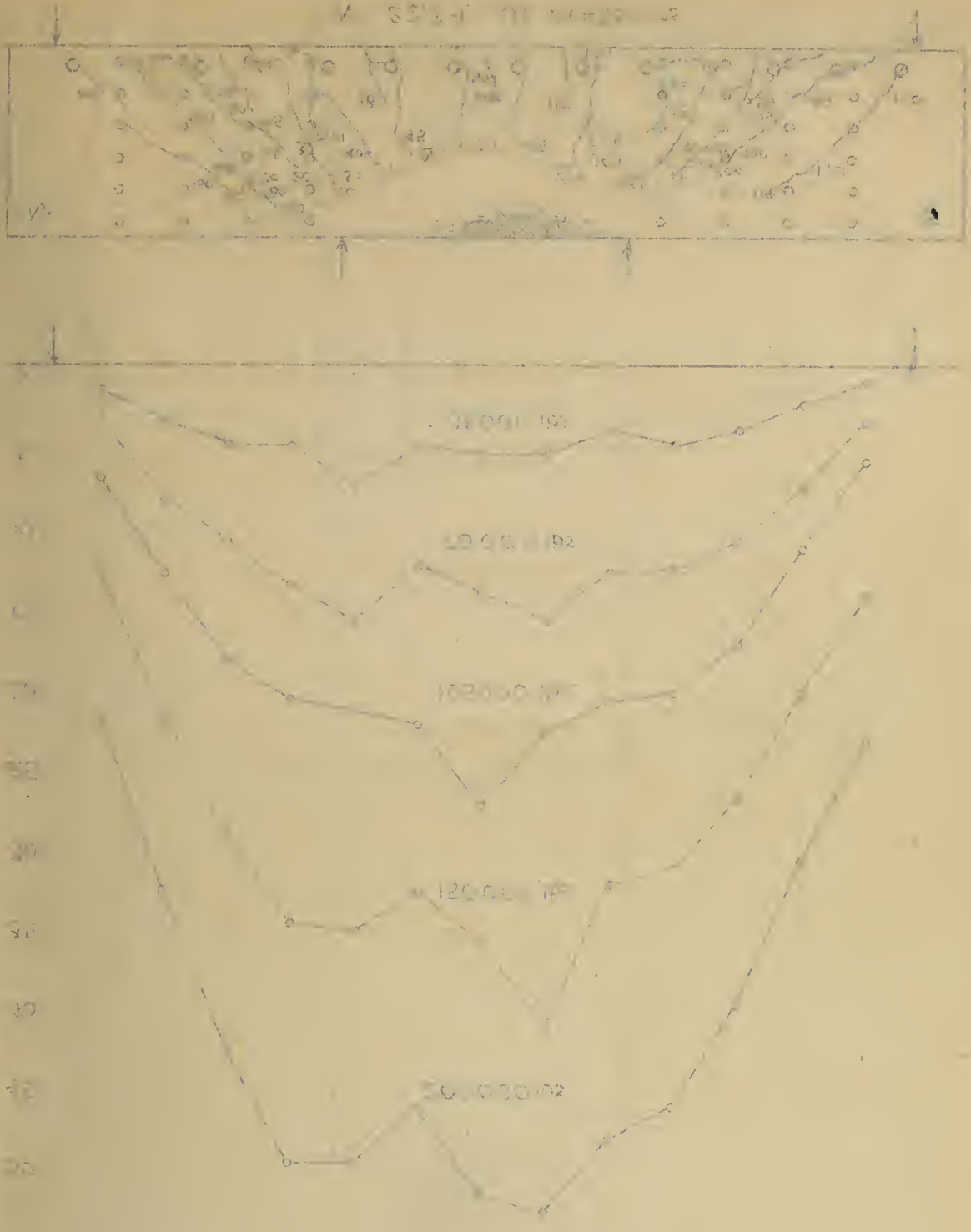

Unit Stress in Thousands of Pounds
40
35
30
25
20
15
10
5
0
140000 lbs.
105000 lbs.
70 000 lbs.
35000 lbs.
N
S
W. 22,2-1 U.L. 167500 lbs.

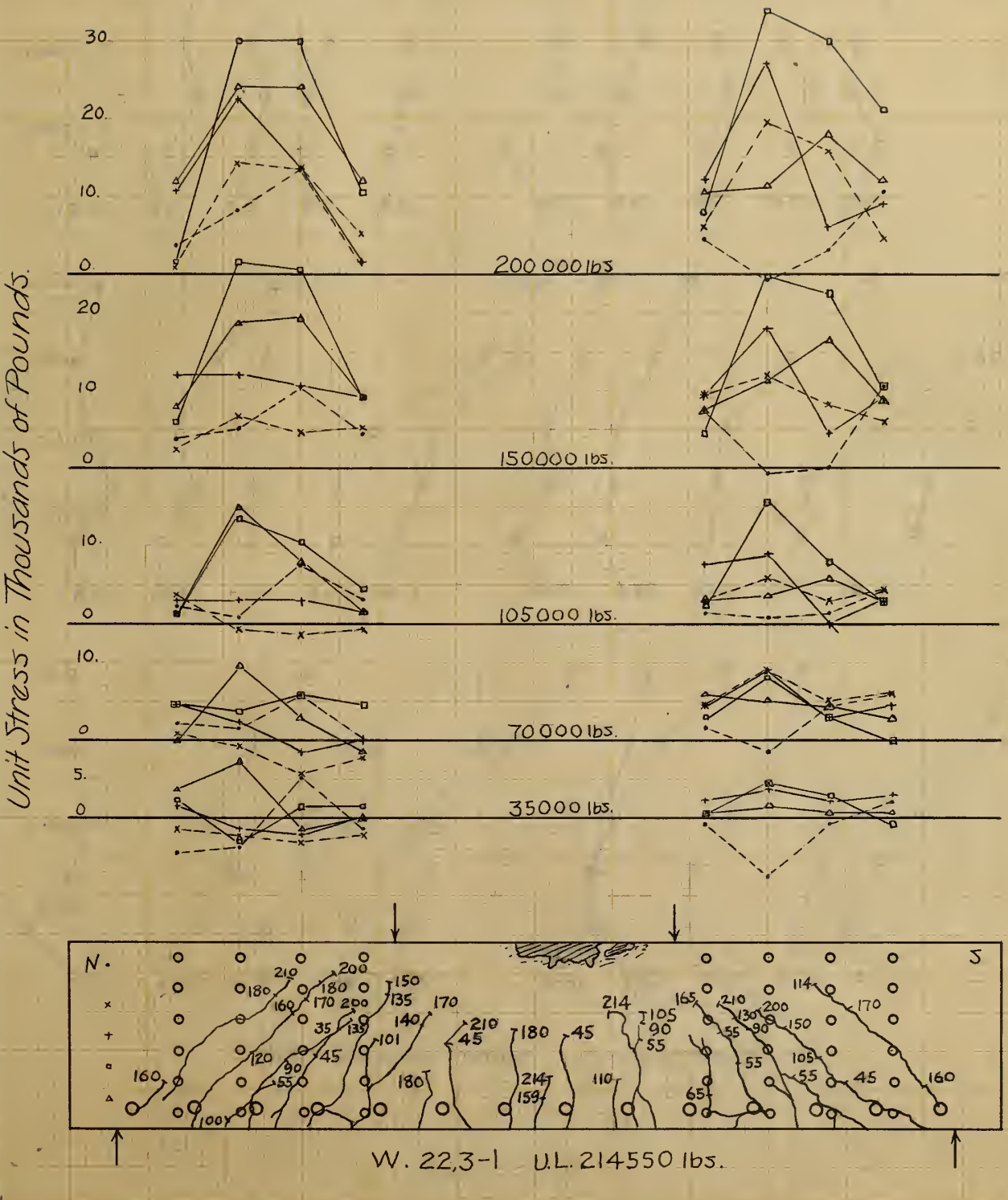

Unit Stress in Thousands of Pounds.
45
40
35
30
25
20
15
10
5
0
105000 lbs.
70000 lbs
35000 lbs.
N
S
W. 22,2-2 U.L. 128000 lbs.

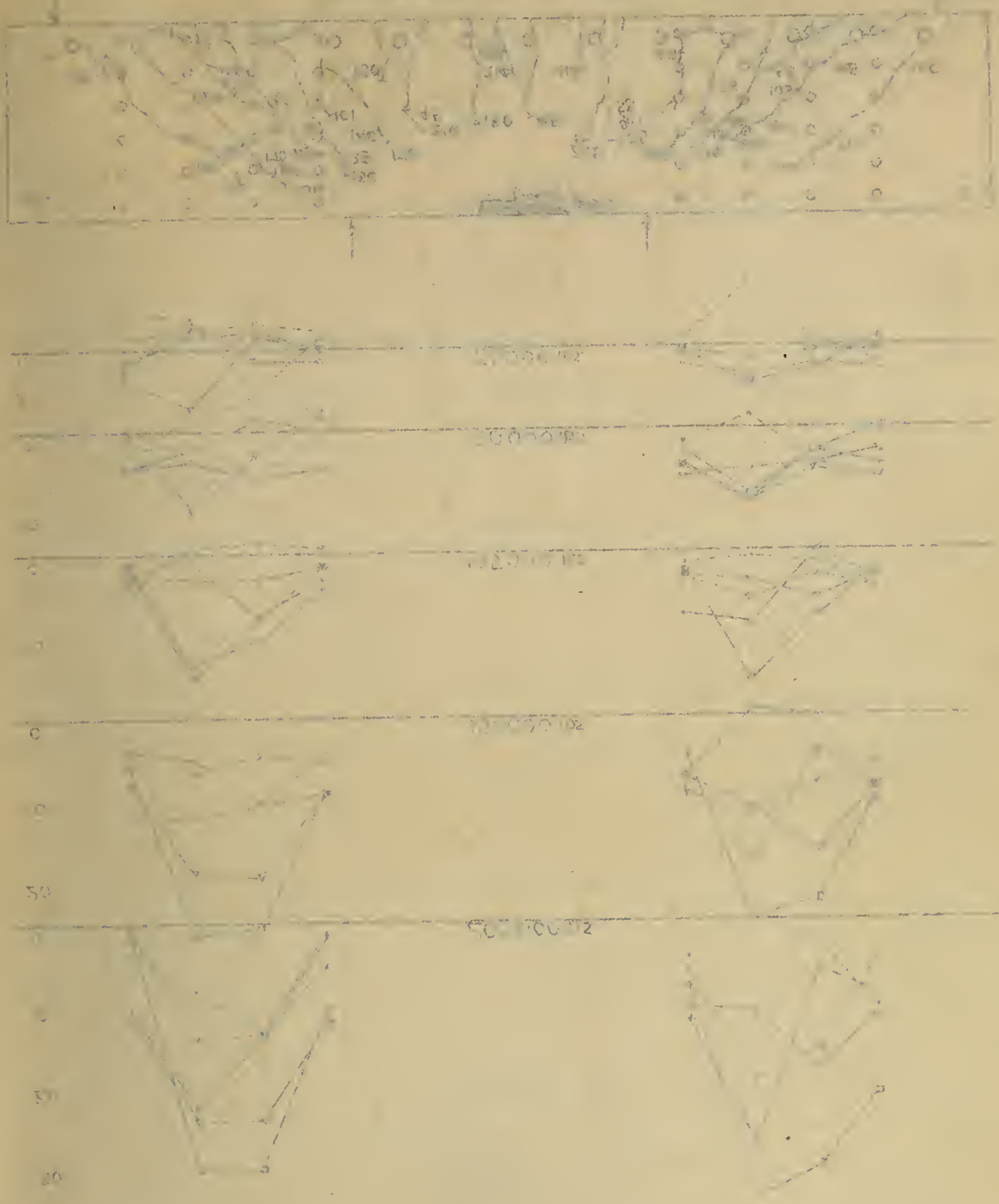
GIRDER MOMENT IN 1000 FOOT POUNDS

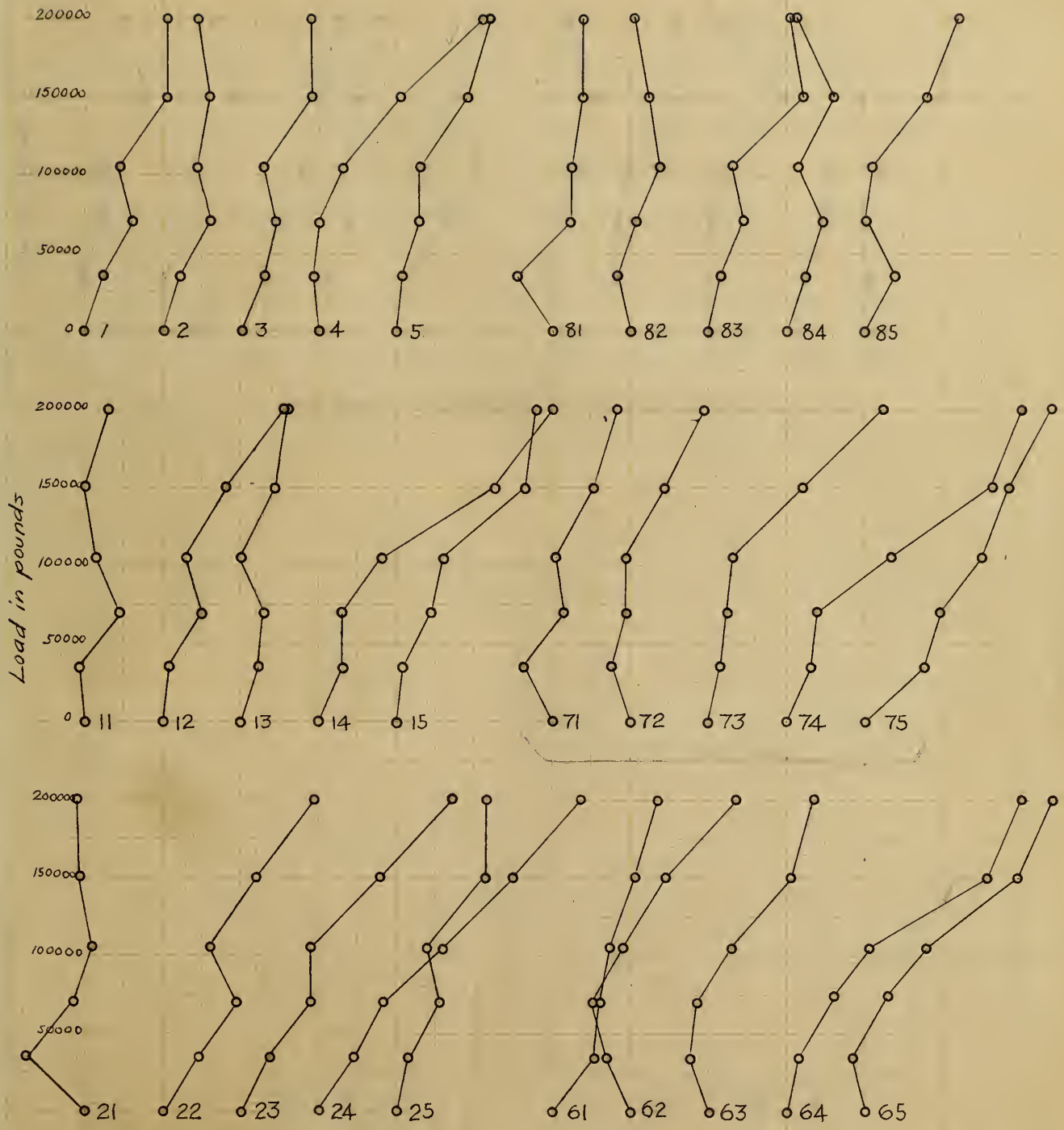

Unit Stress in Thousands of Pounds.
35
30
25
20
15
10
5
0
105000 lbs.
70000 lbs.
35000 lbs.
S
N
E 22,2-2 U.L. 128000 lbs.


Unit Stress in Thousands of Pounds.
55
50
45
40
35
30
25
20
15
10
5
0
200000 lbs.
150000 lbs
105000 lbs.
70000 lbs.
35000 lbs.
N
S
W. 22,3-1. U.L. 214550 lbs.



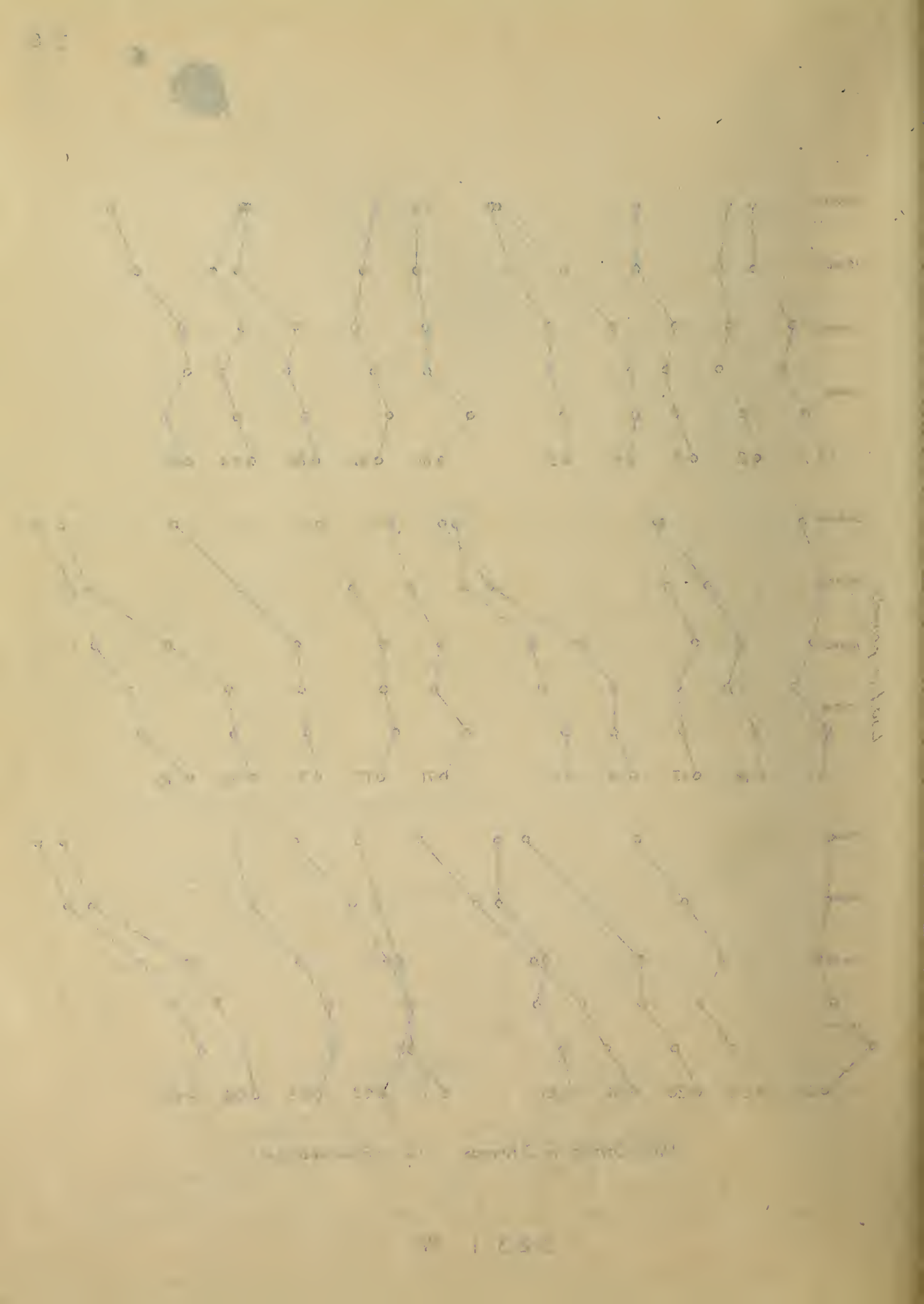

Unit Stress in Thousands of Pounds.
30.
20.
10.
0.
200 000 lbs.
20
10
0
150000 lbs.
10.
0
105000 lbs.
10.
0
70000 lbs.
5.
0
35000 lbs.
N.
S
W. 22,3-1 U.L. 214550 lbs.



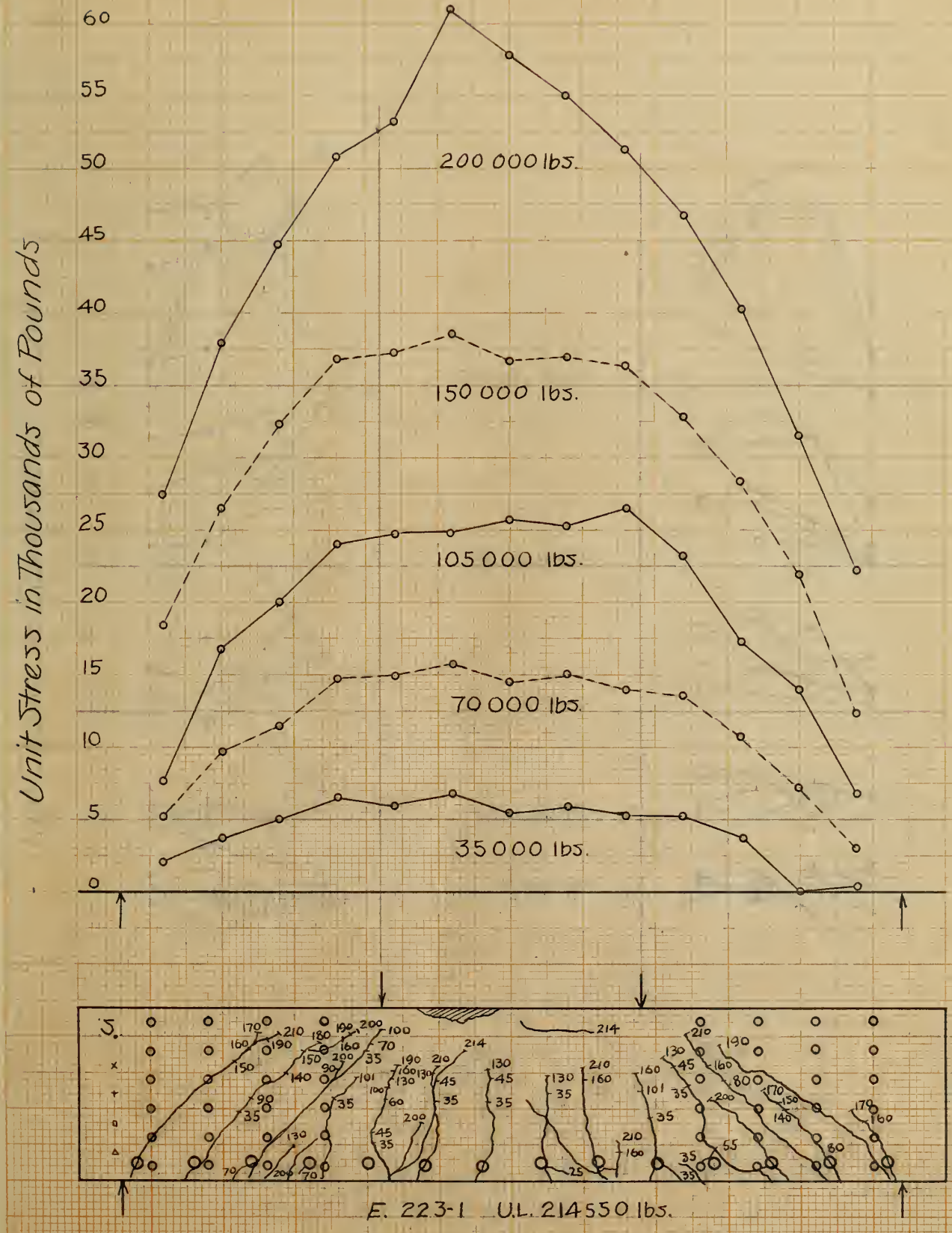


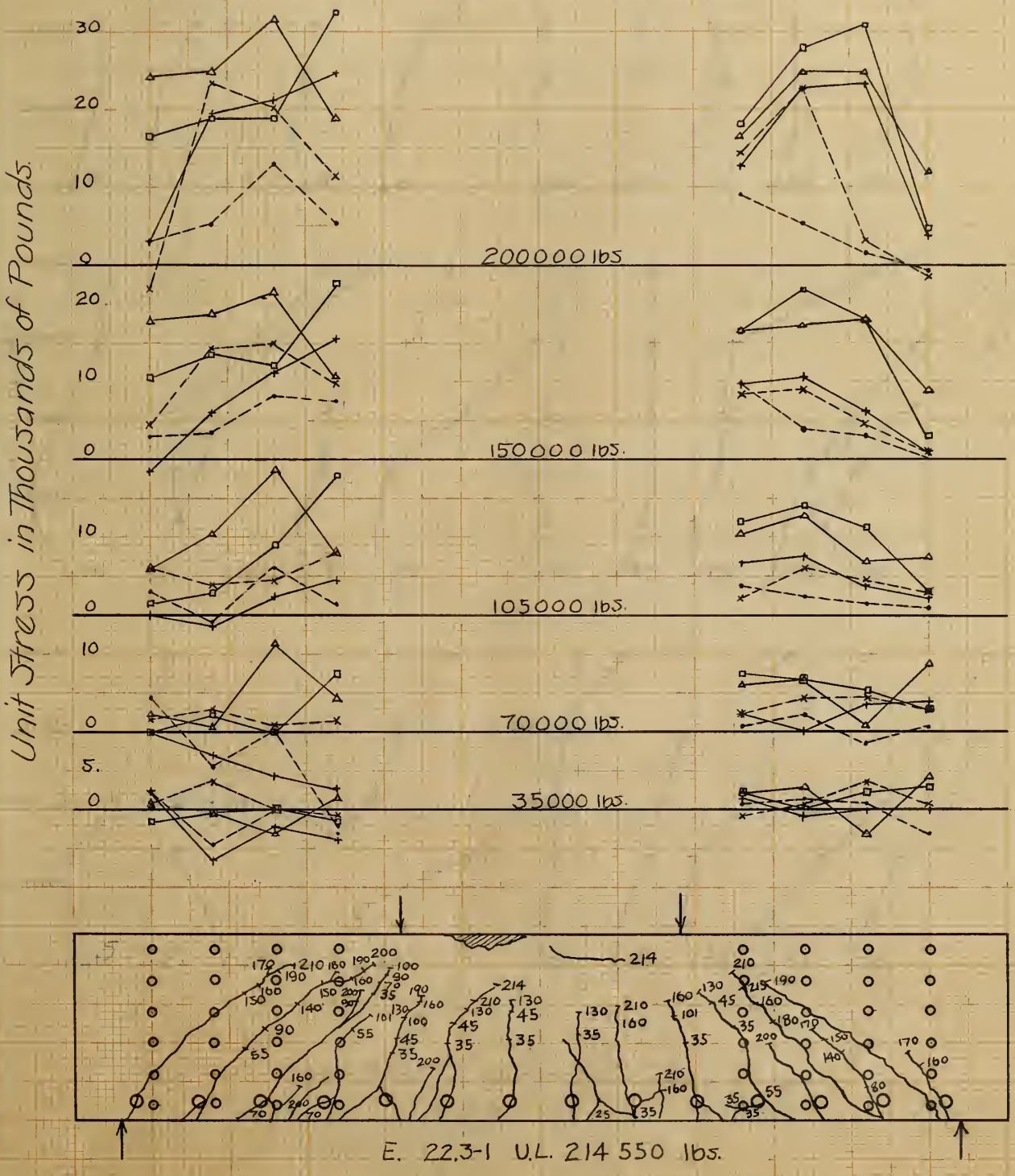
Unit Stress in Stirrups 1 in = 20000 lb per sq in.

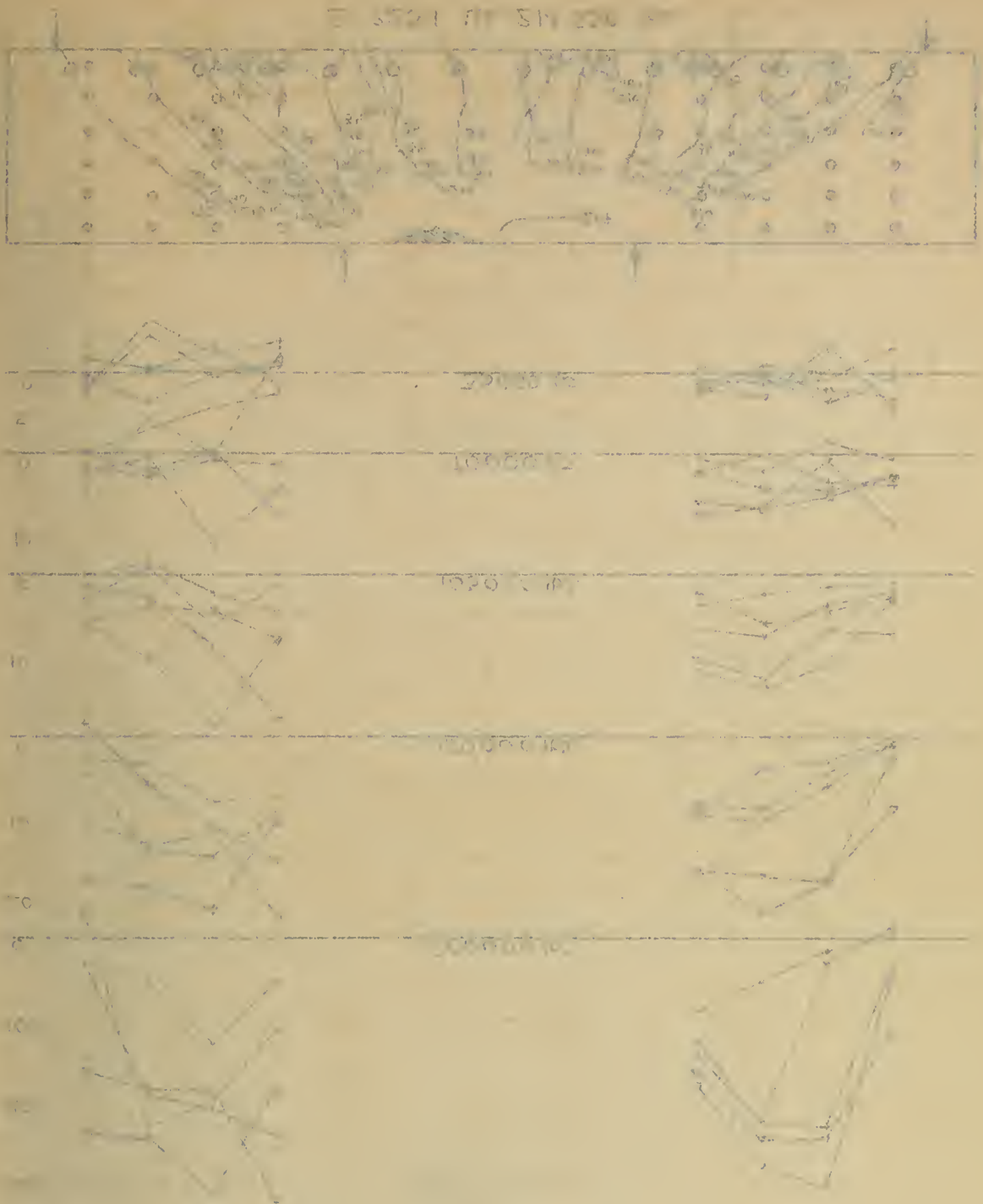
22,3-1 W

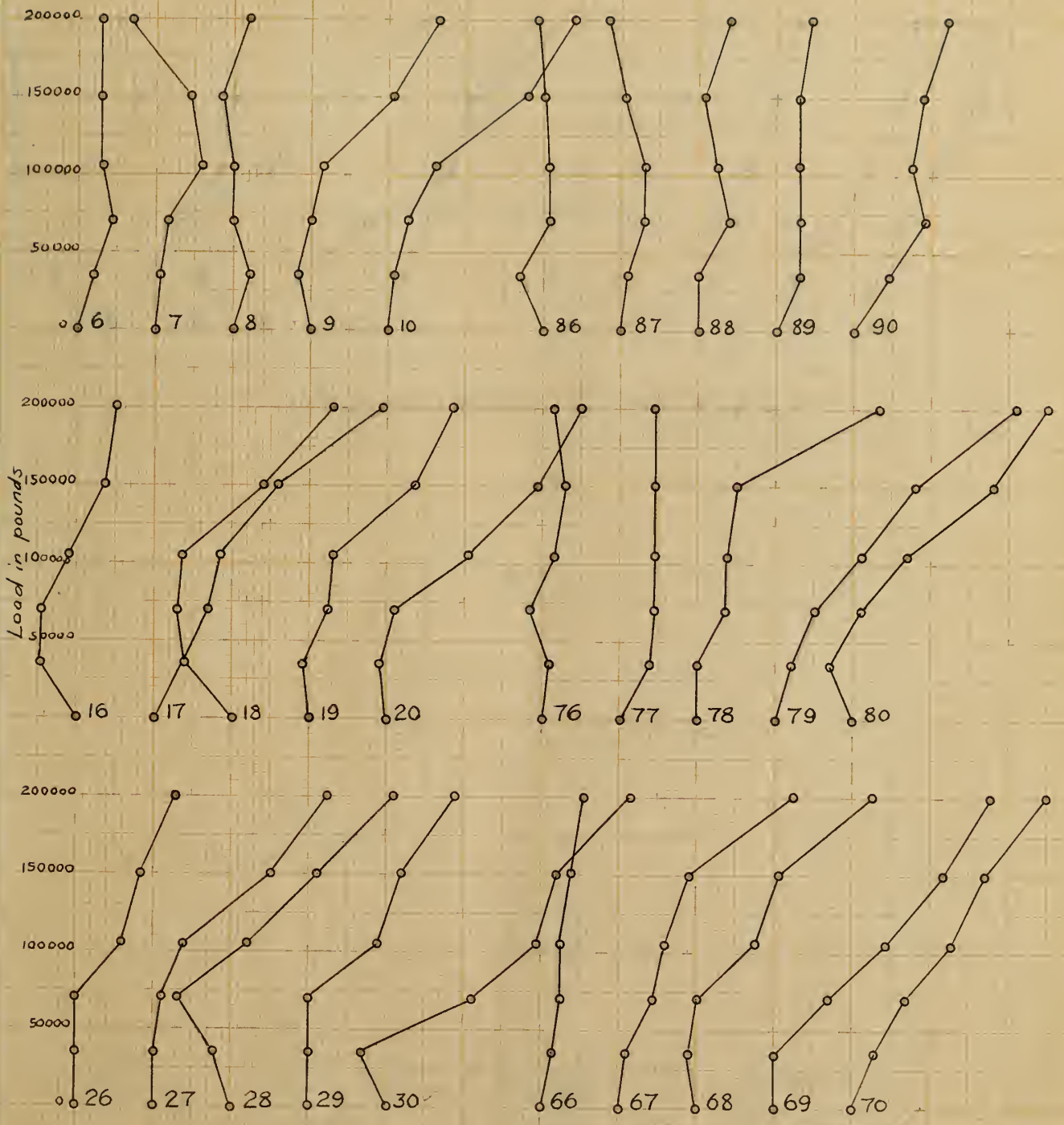


Unit Stress in Stirrups 1 in = 20000 lb. per sq. in.


Unit Stress in Thousands of Pounds
60
55
50
45
40
35
30
25
20
15
10
5
0
200 000 lbs.
150 000 lbs.
105 000 lbs.
70 000 lbs.
35 000 lbs.
E. 223-1 U.L. 214550 lbs.

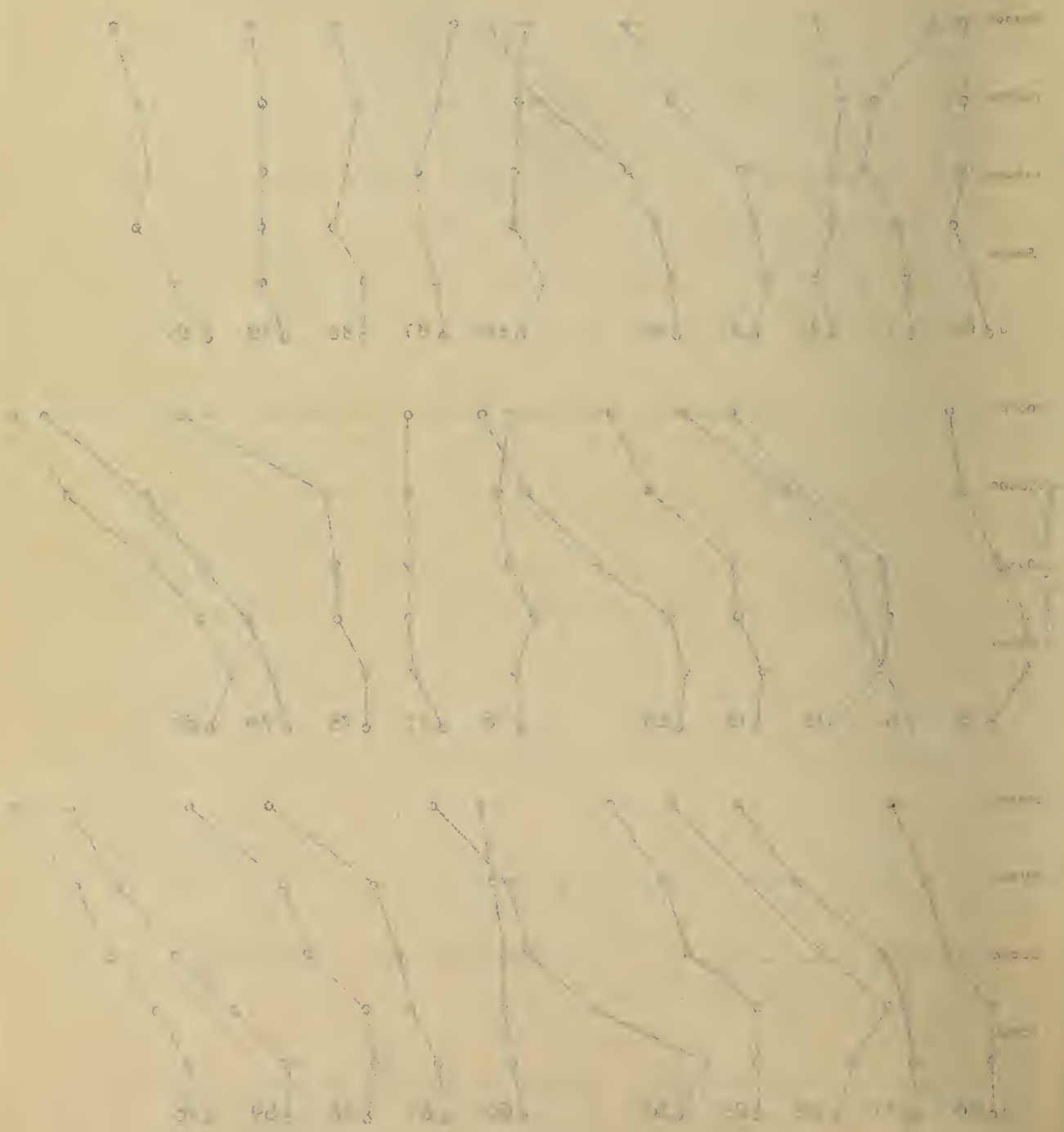

Unit Stress in Thousands of Pounds
30
20
10
0
200000 lbs.
20
10
0
150000 lbs.
10
0
105000 lbs.
10
0
70000 lbs.
5.
0
35000 lbs.
E. 22.3-1 U.L. 214 550 lbs.

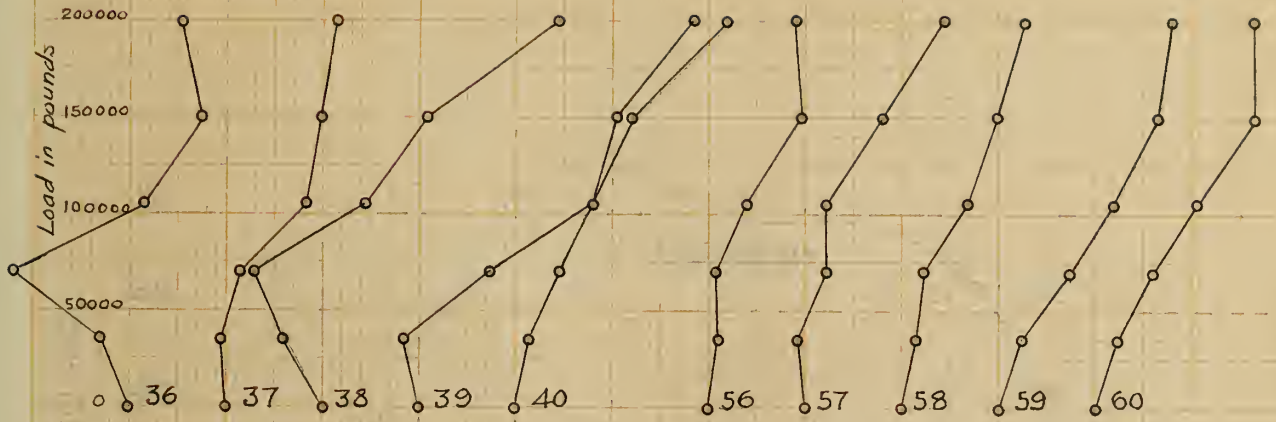




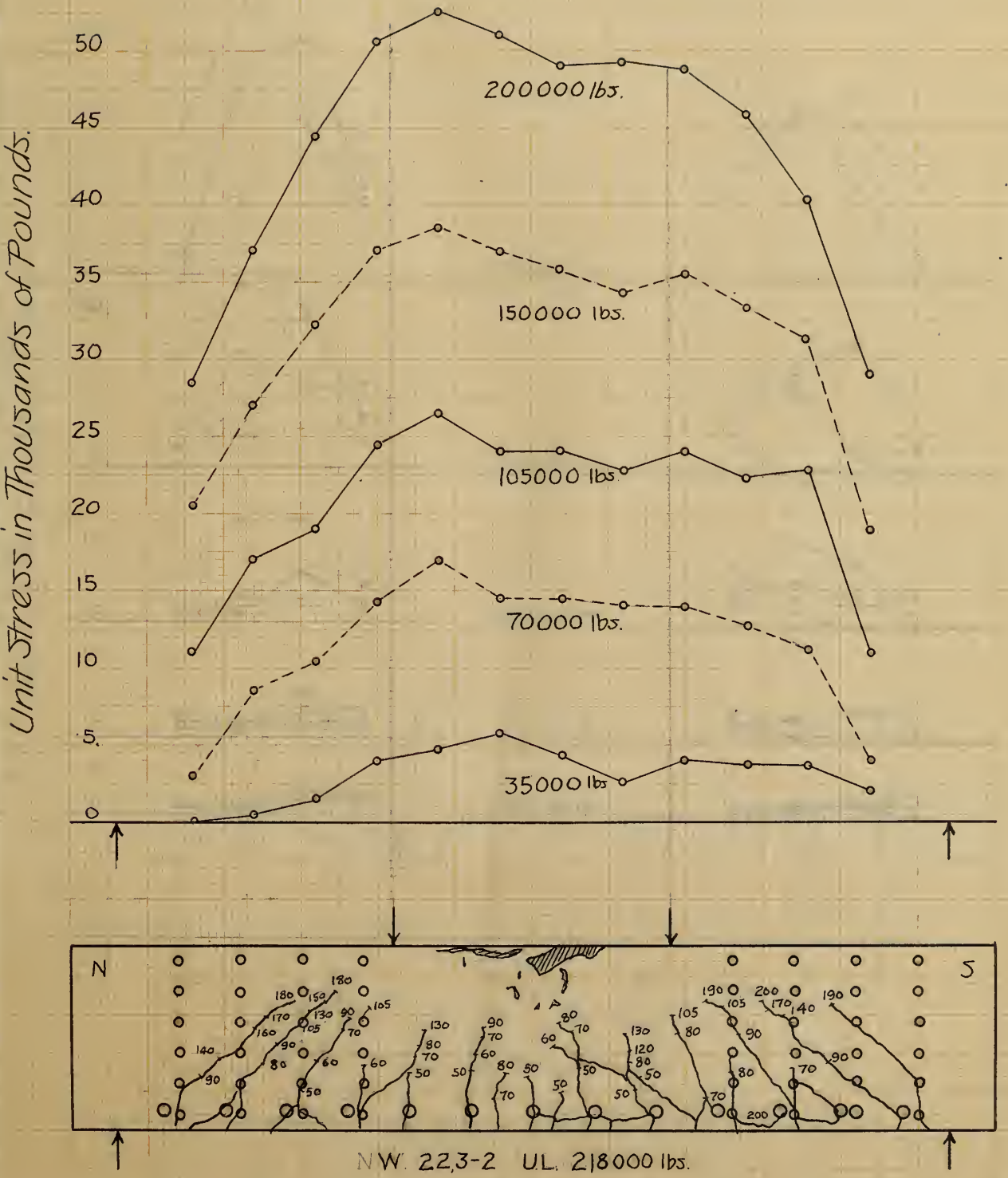
Unit Stress in Stirrups. 1 in. = 20000 lb. per sq. in.

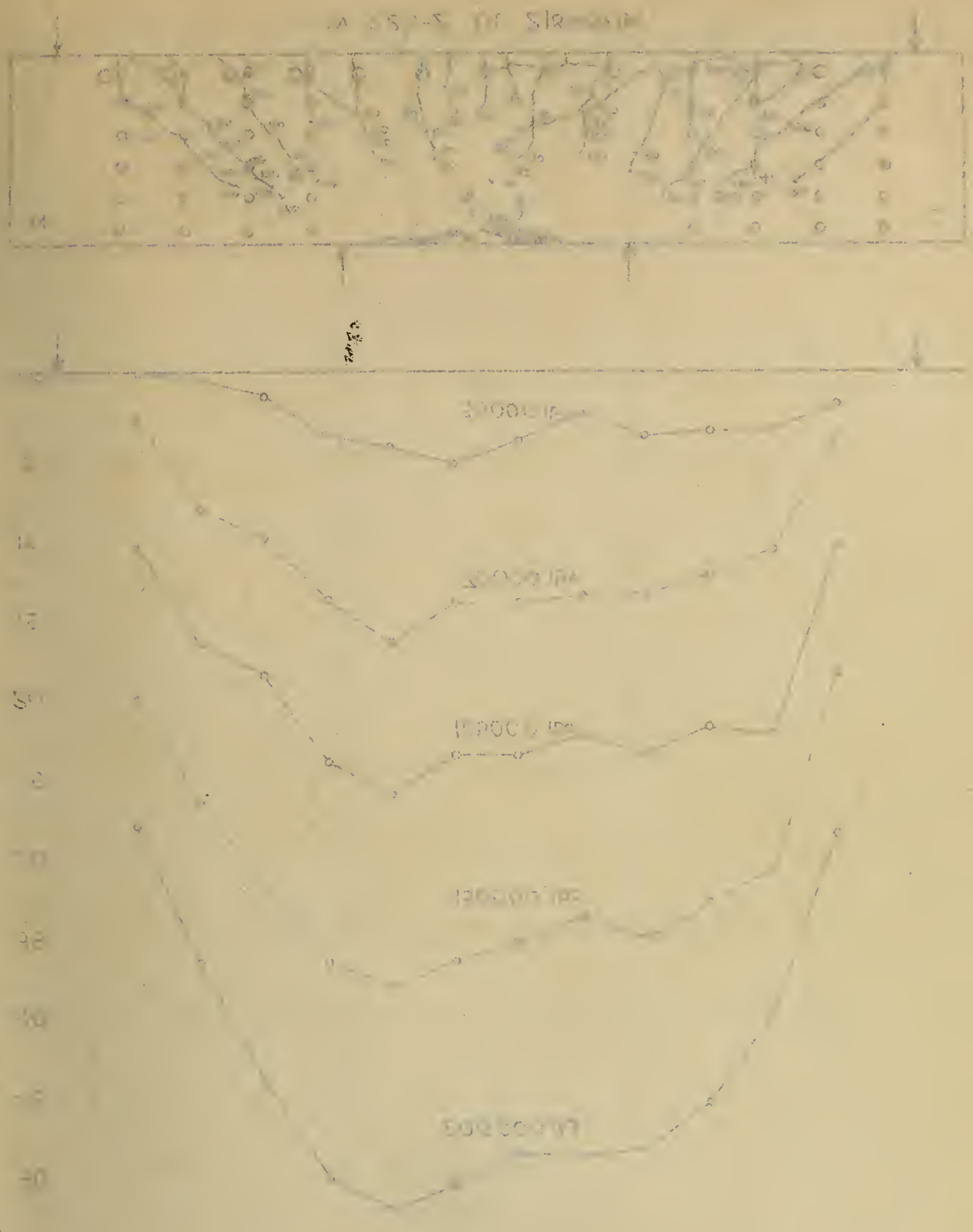
22,3-1 E.

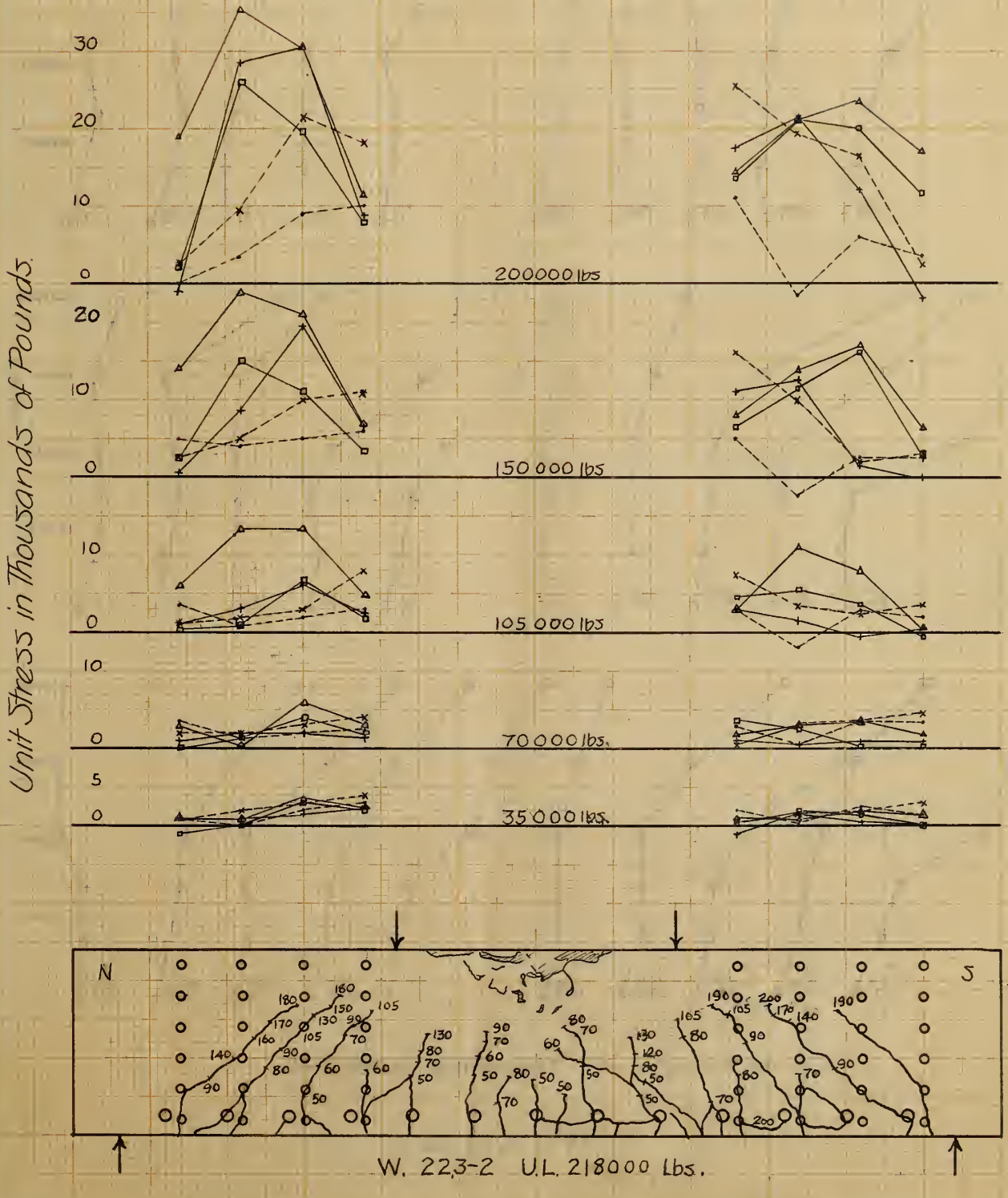


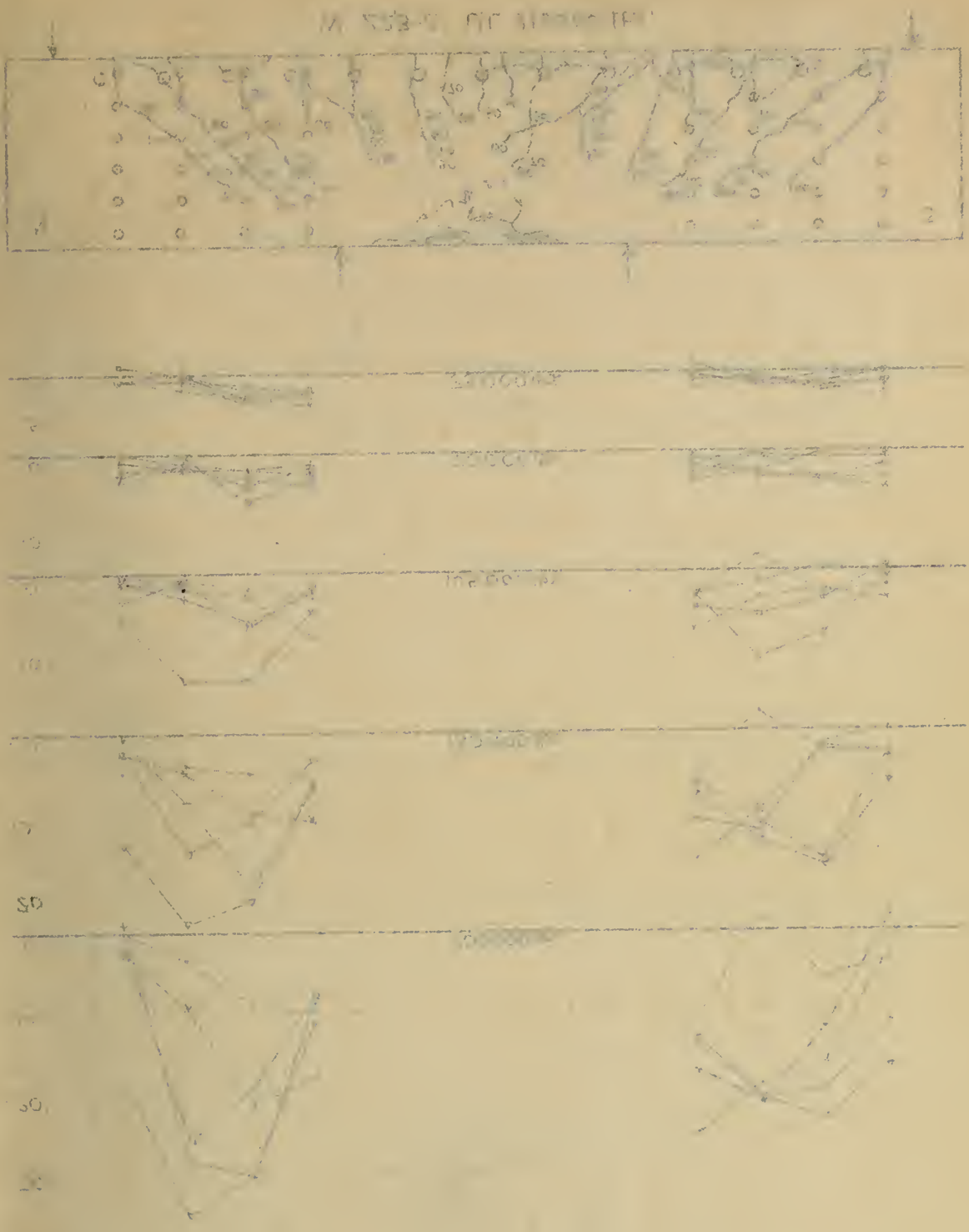


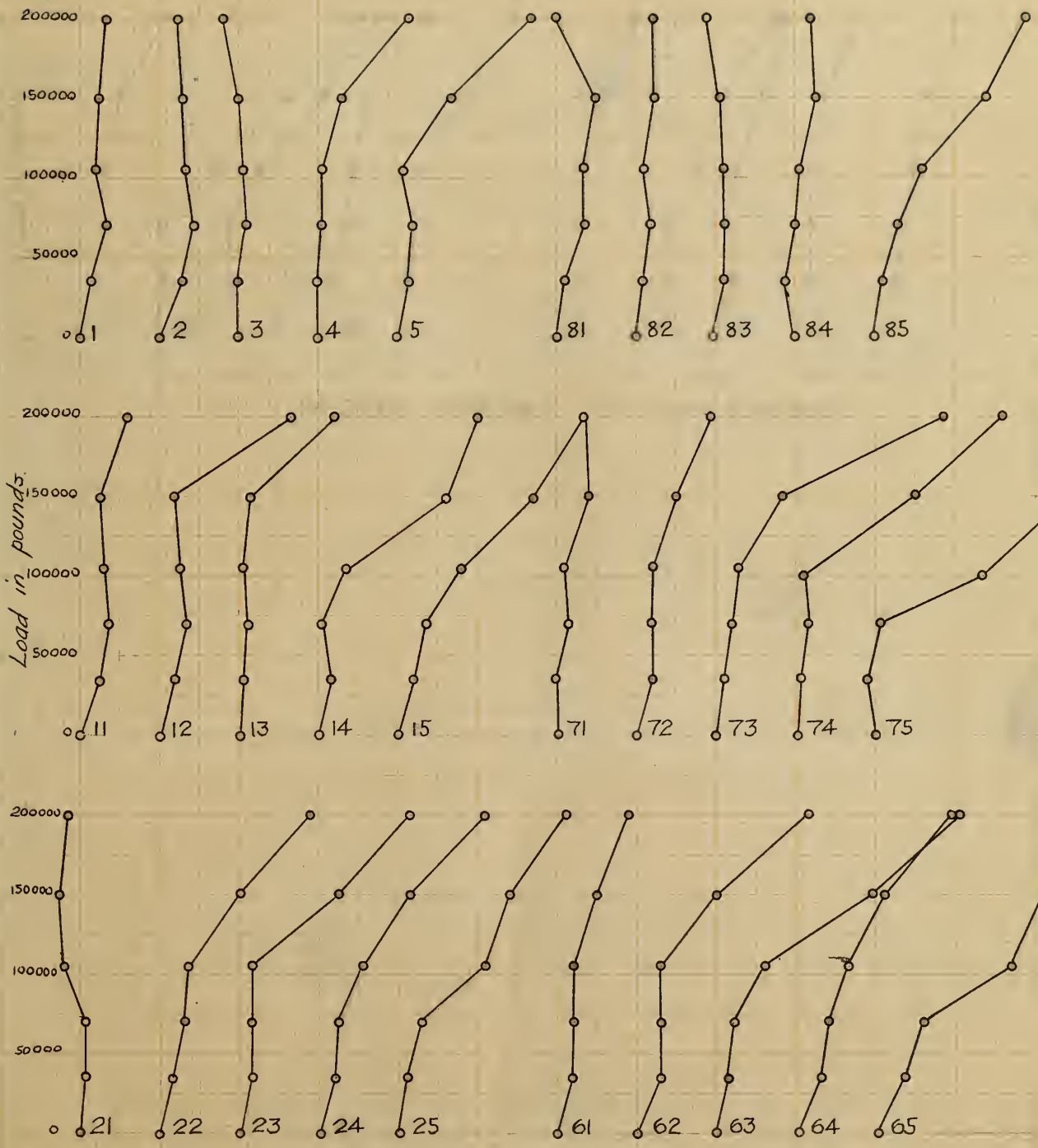
Unit Stress in Stirrups. 1 in = 20000 lb. per sq. in.


Unit Stress in Thousands of Pounds.
50
45
40
35
30
25
20
15
10
5
0
200000 lbs.
150000 lbs.
105000 lbs.
70000 lbs.
35000 lbs
N
S
NW. 22,3-2 UL. 218000 lbs.




Unit Stress in Thousands of Pounds.
30
20
10
0
200000 lbs
20
10
0
150 000 lbs
10
0
105 000 lbs
10
0
70 000 lbs.
5
0
35 000 lbs.
N
S
W. 22,3-2 U.L. 218000 Lbs.





Unit Stress in Stirrups 1 in. = 20000 lb. per sq. in.

22,3-2. W.

Unit Stress in Stirrups. 1 in. = 20000 lb. per sq. in.

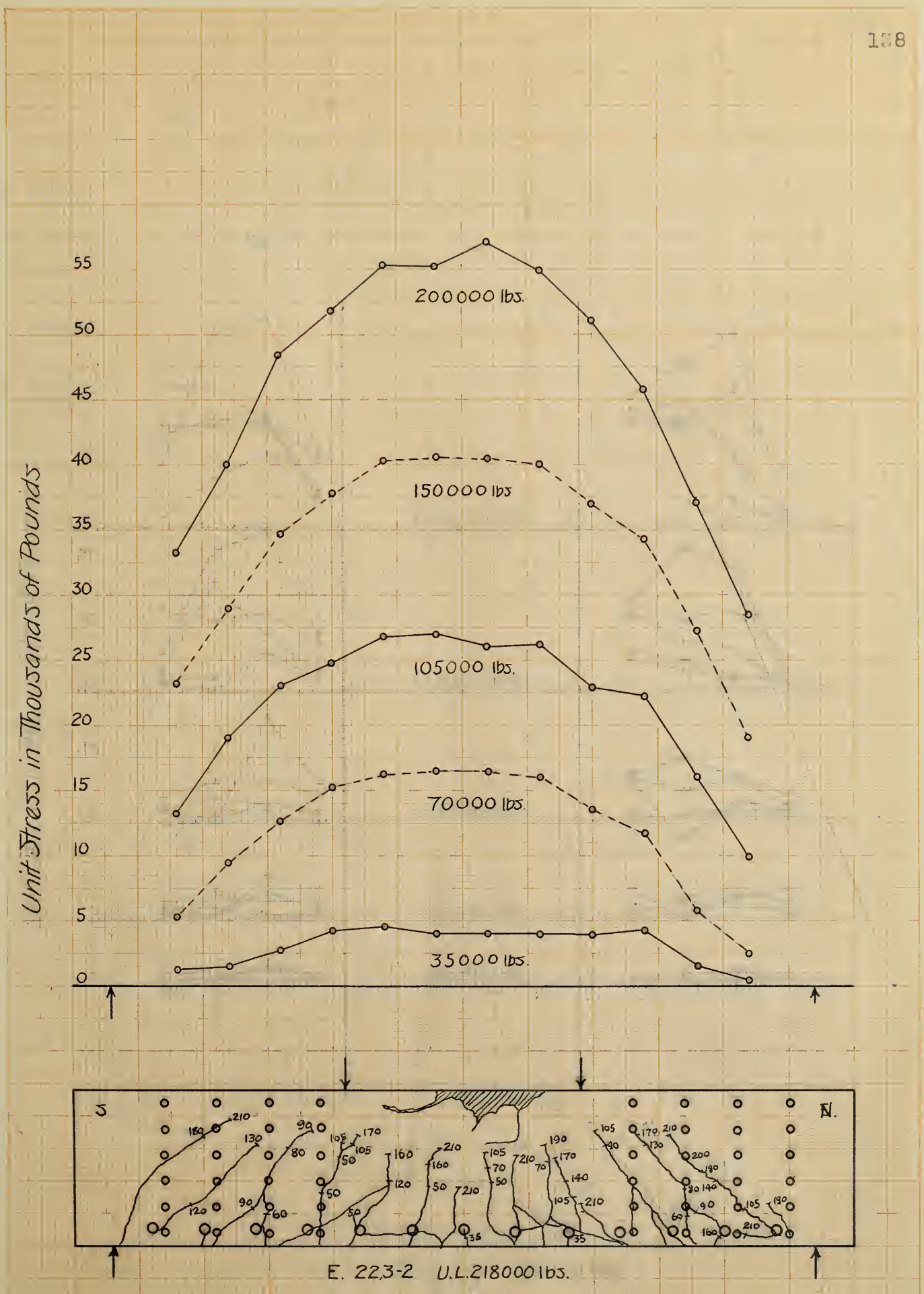
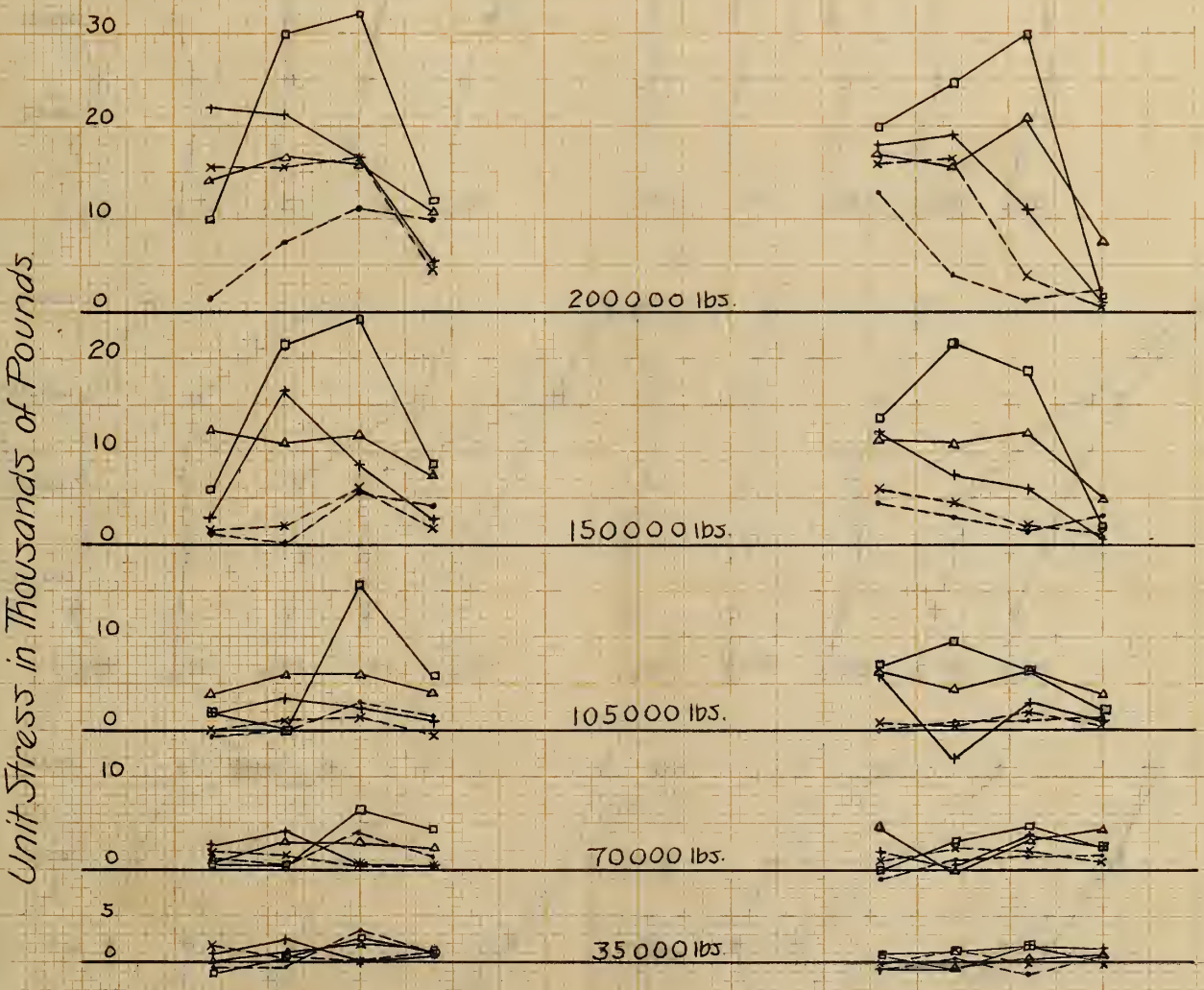
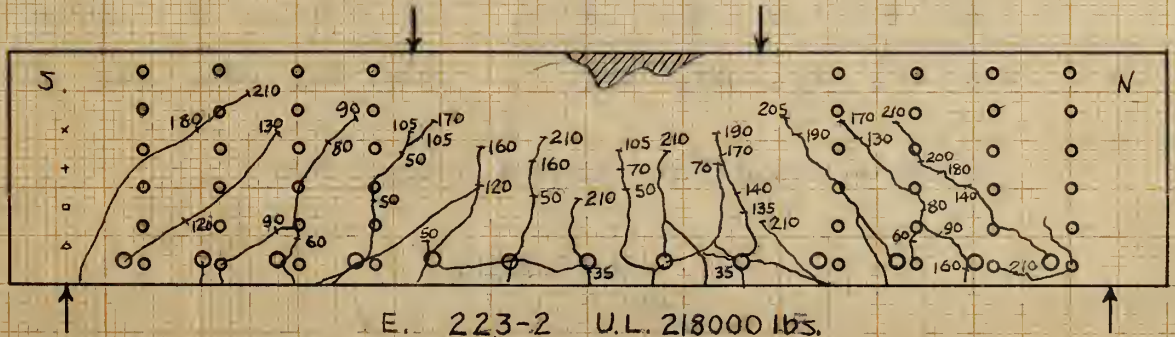
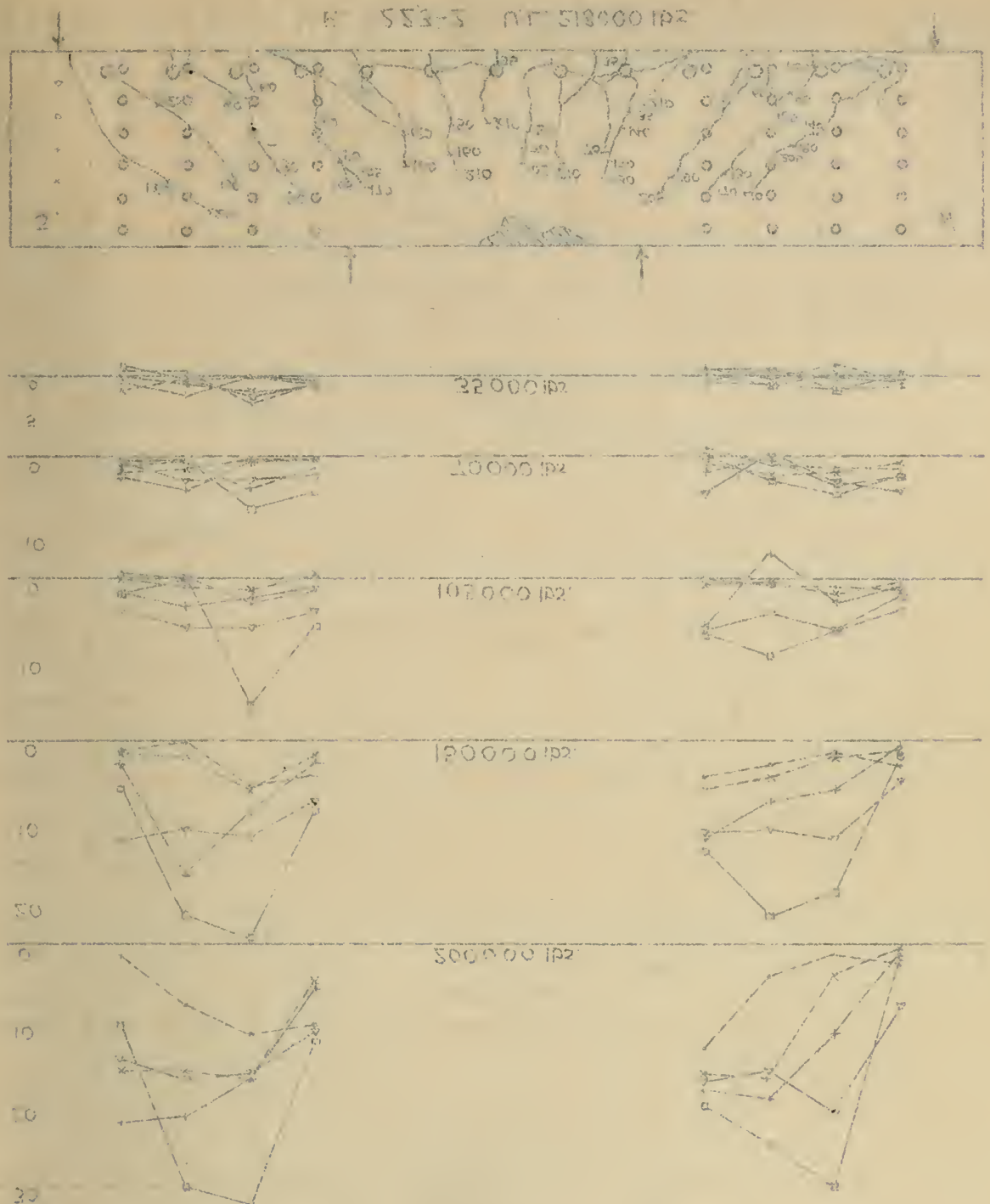

Unit Stress in Thousands of Pounds
55
50
45
40
35
30
25
20
15
10
5
0
200000 lbs.
150000 lbs
105000 lbs.
70000 lbs.
35000 lbs.
S
N.
E. 223-2 U.L. 218000 lbs.

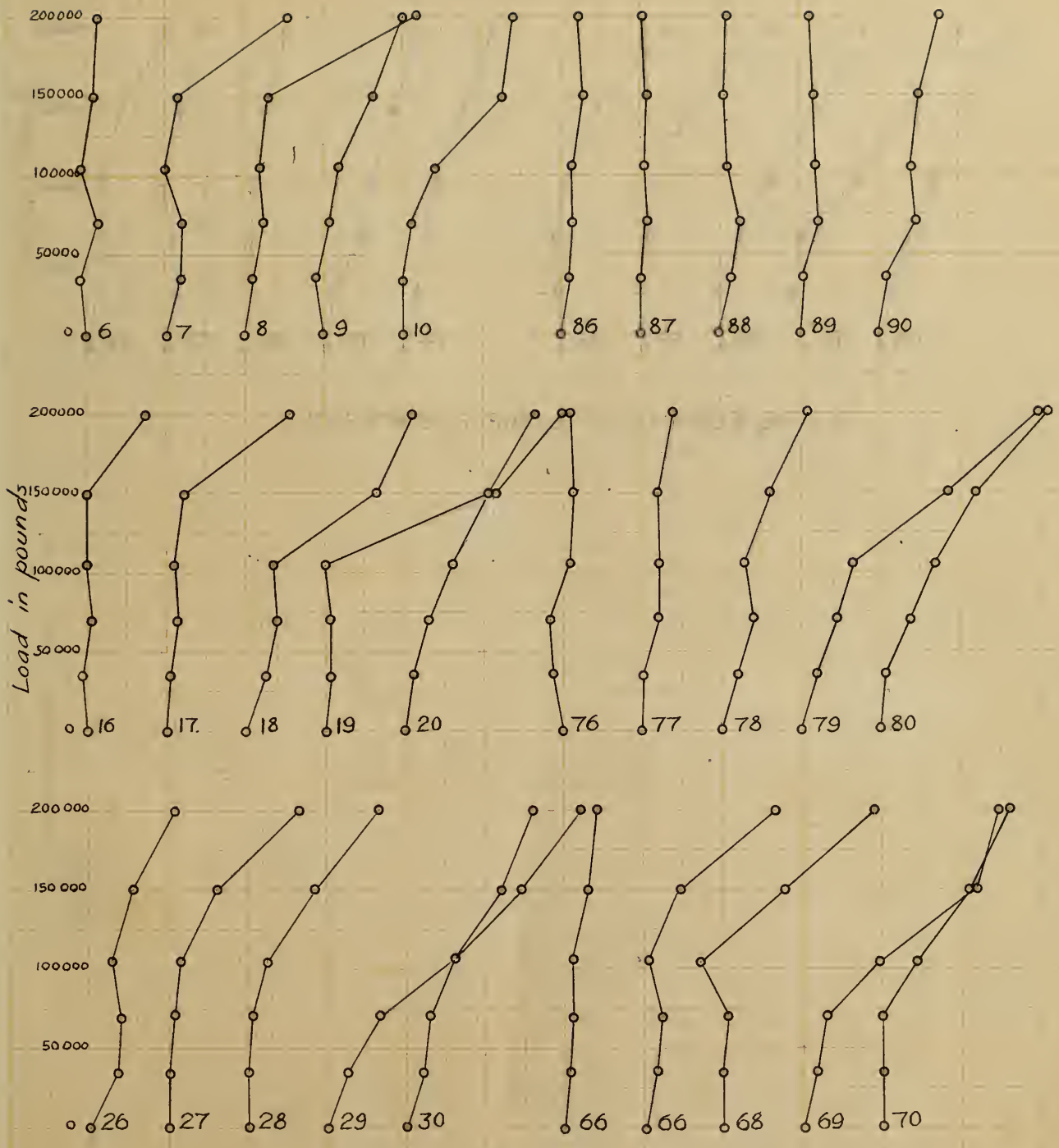
Diagram of Pressure on Bridge Piers





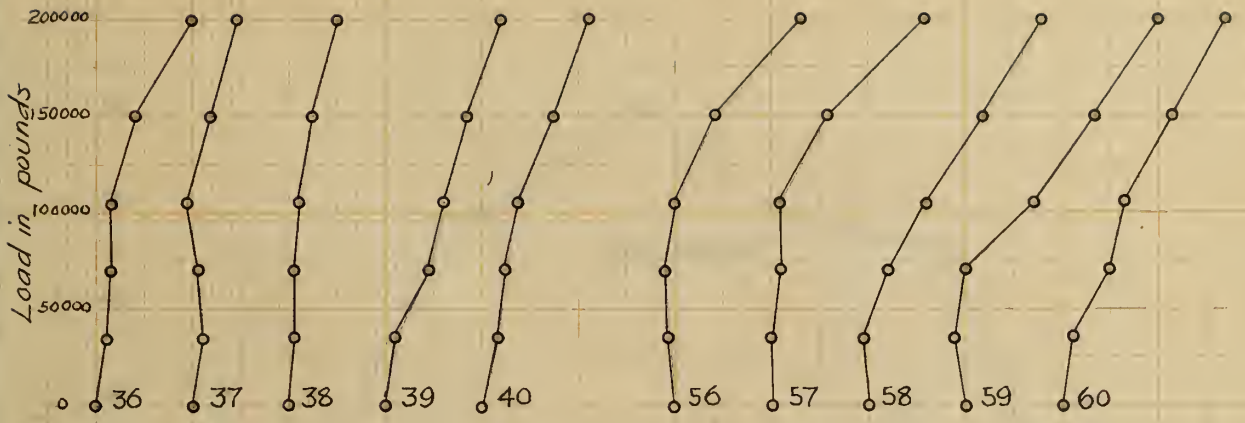
E. 223-2 U.L. 218000 lbs.



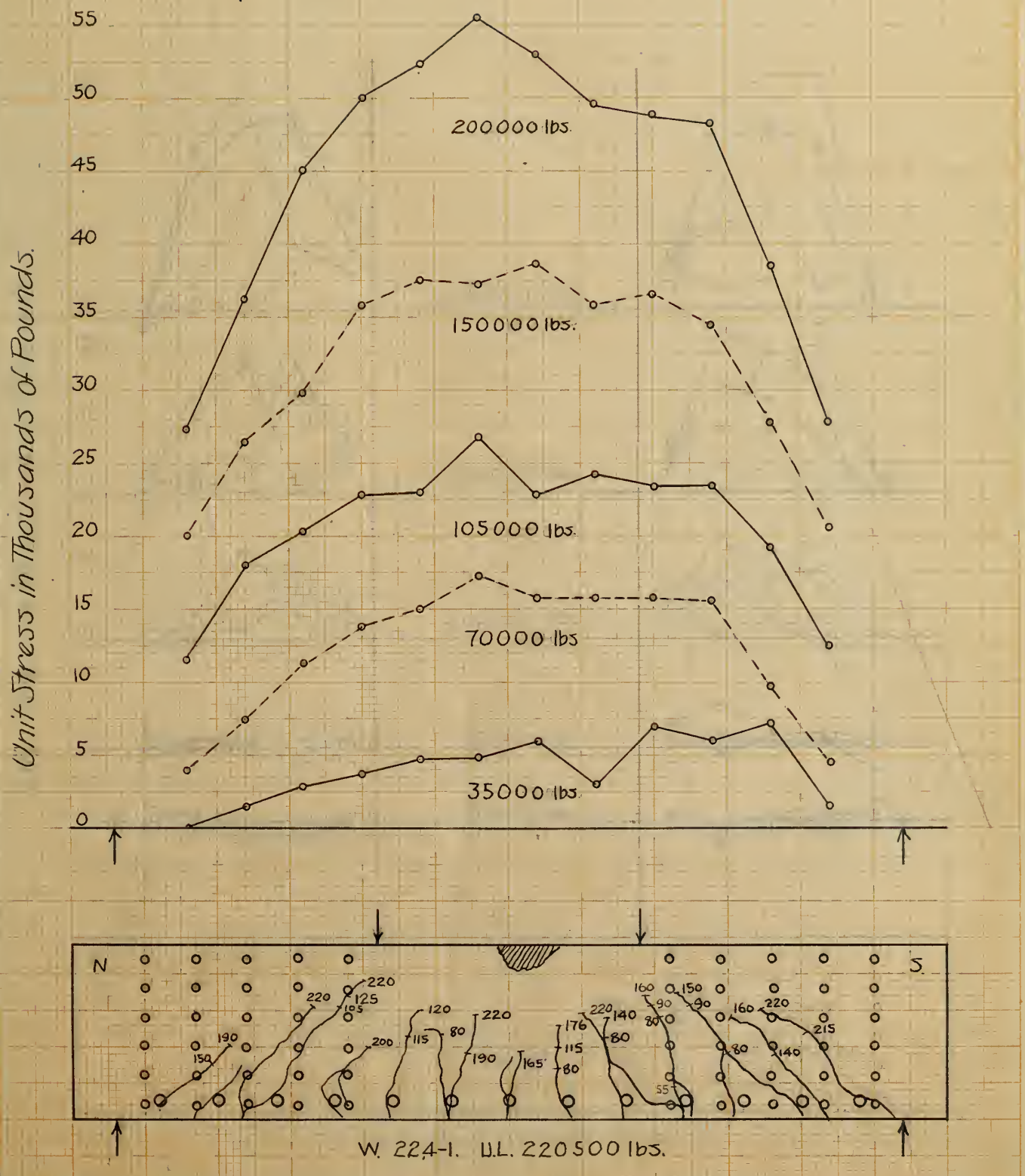


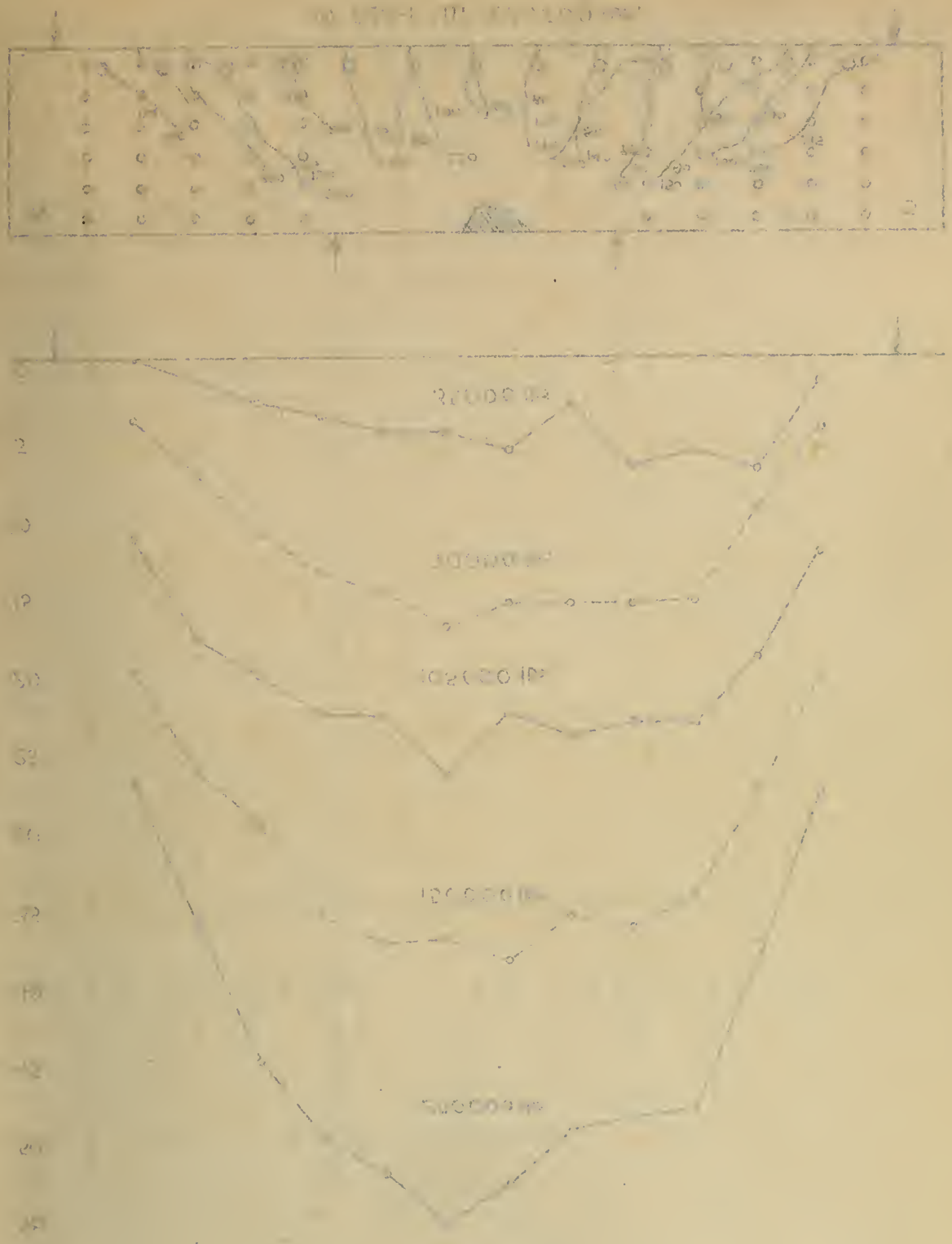
Unit Stress in Stirrups. 1 in. = 20000 lb. per sq. in.

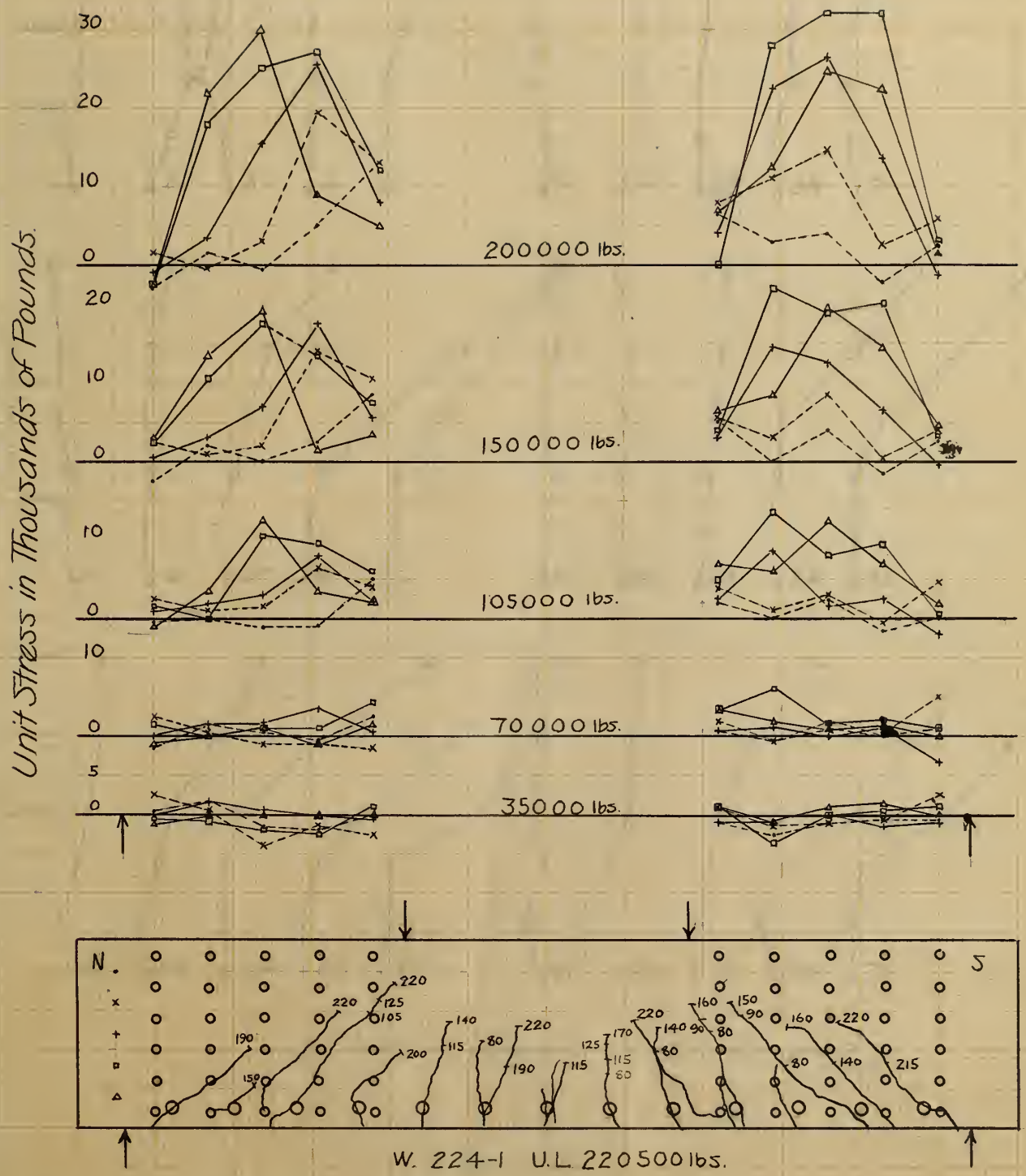
22,3-2 E.

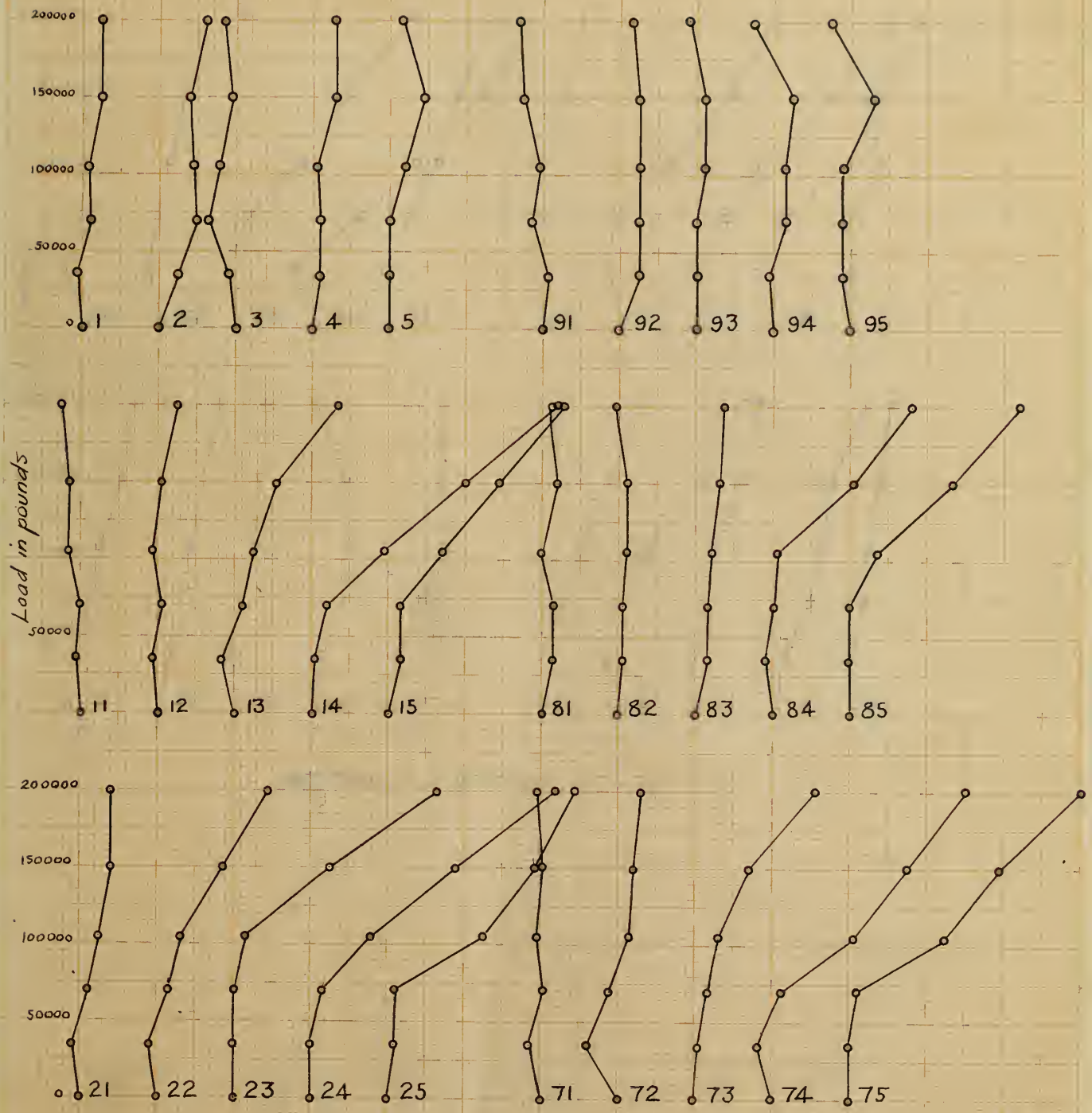


Unit-Stress in Stirrups. 1 in. = 20000 lb. per sq. in.


Unit Stress in Thousands of Pounds.
55
50
45
40
35
30
25
20
15
10
5
0
200000 lbs.
150000 lbs.
105000 lbs.
70000 lbs.
35000 lbs.
N
S
W. 224-1. U.L. 220500 lbs.

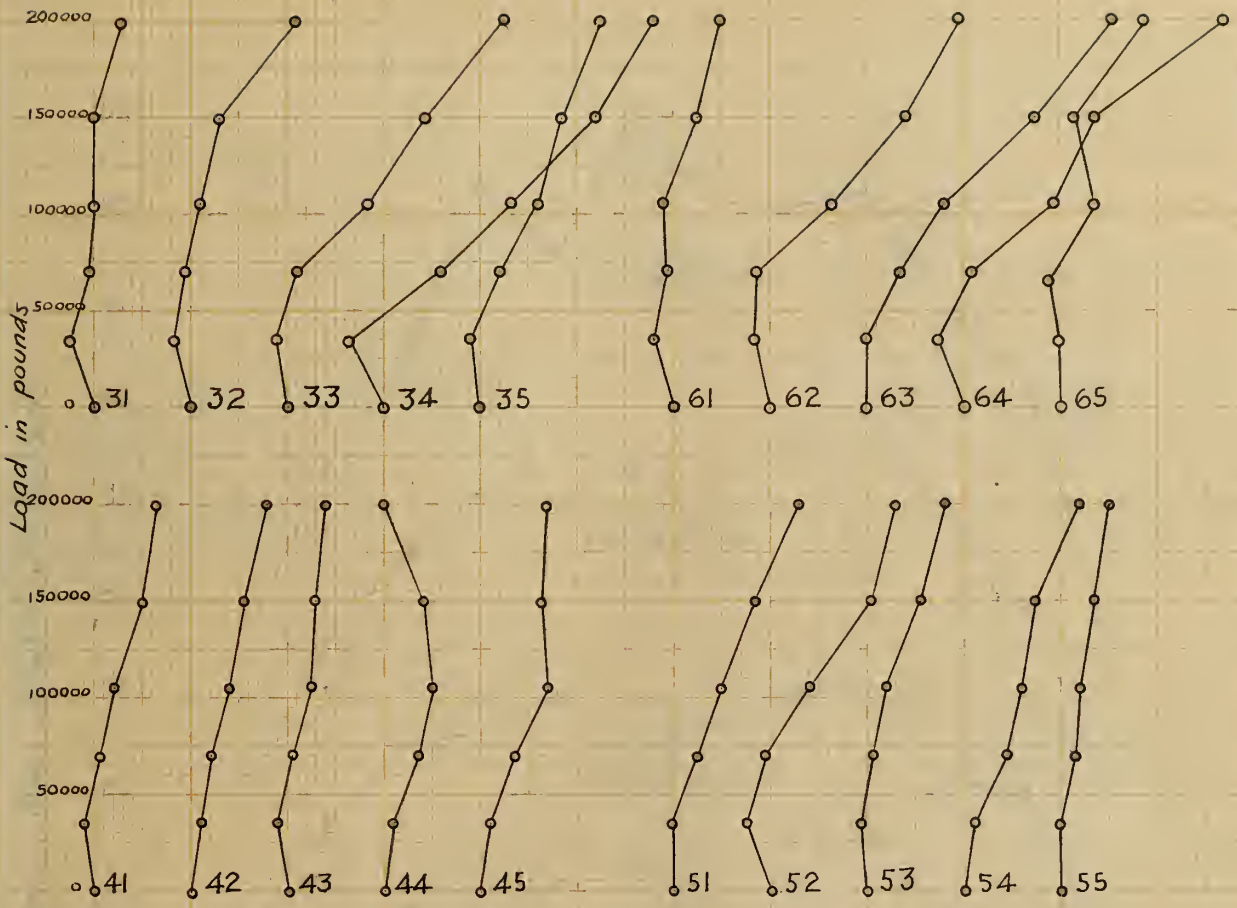



Unit Stress in Thousands of Pounds.
30
20
10
0
200000 lbs.
20
10
0
150000 lbs.
10
0
105000 lbs.
10
0
70000 lbs.
5
0
35000 lbs.
N.
S
220
220
125
105
190
159
200
140
115
80
220
190
115
125
170
115
80
220
140
90
80
160
150
90
80
160
220
80
140
215
W. 224-1 U.L. 220500 lbs.

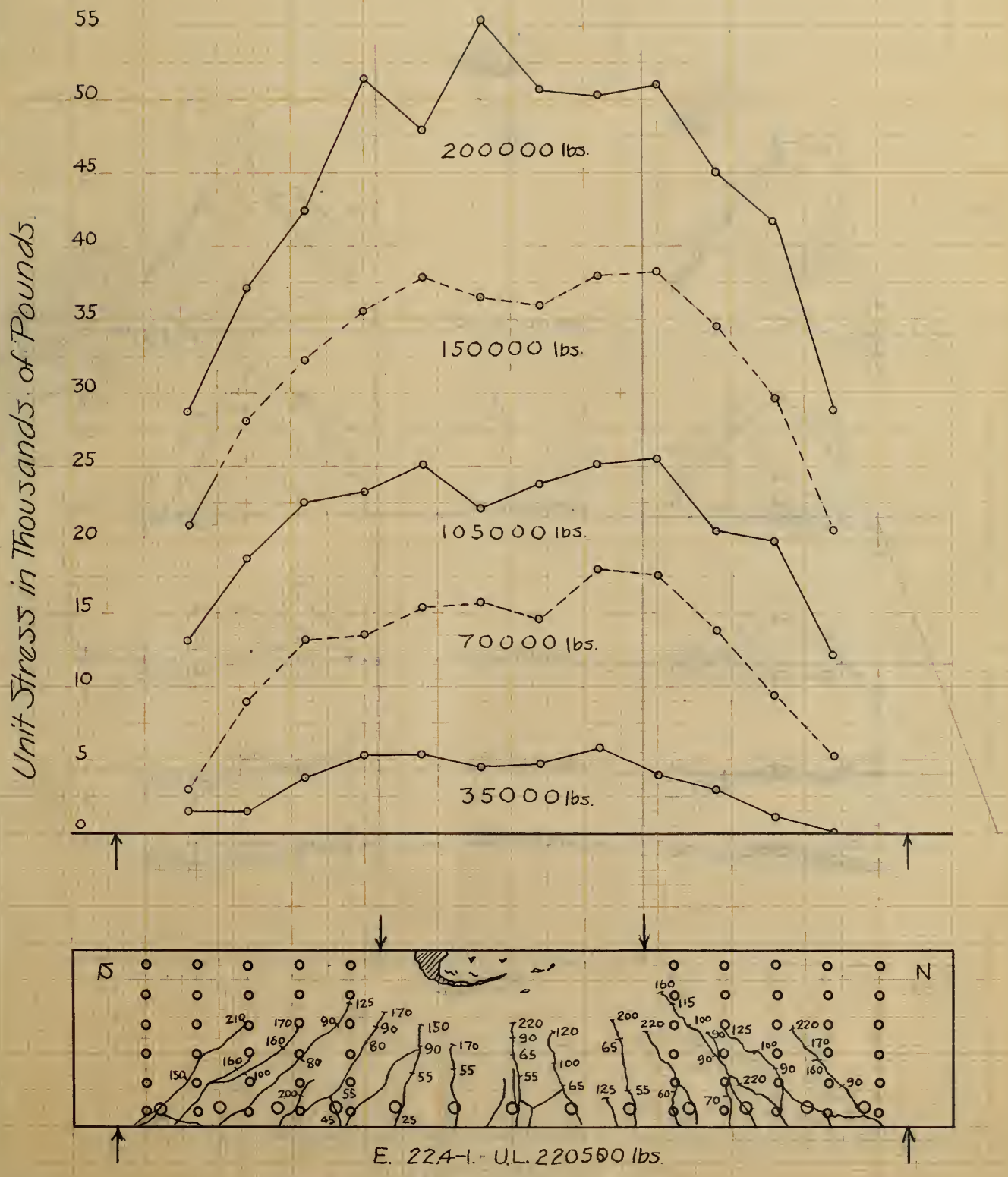


Unit Stress in Stirrups. 1 in. = 20 000 lb. per sq. in.

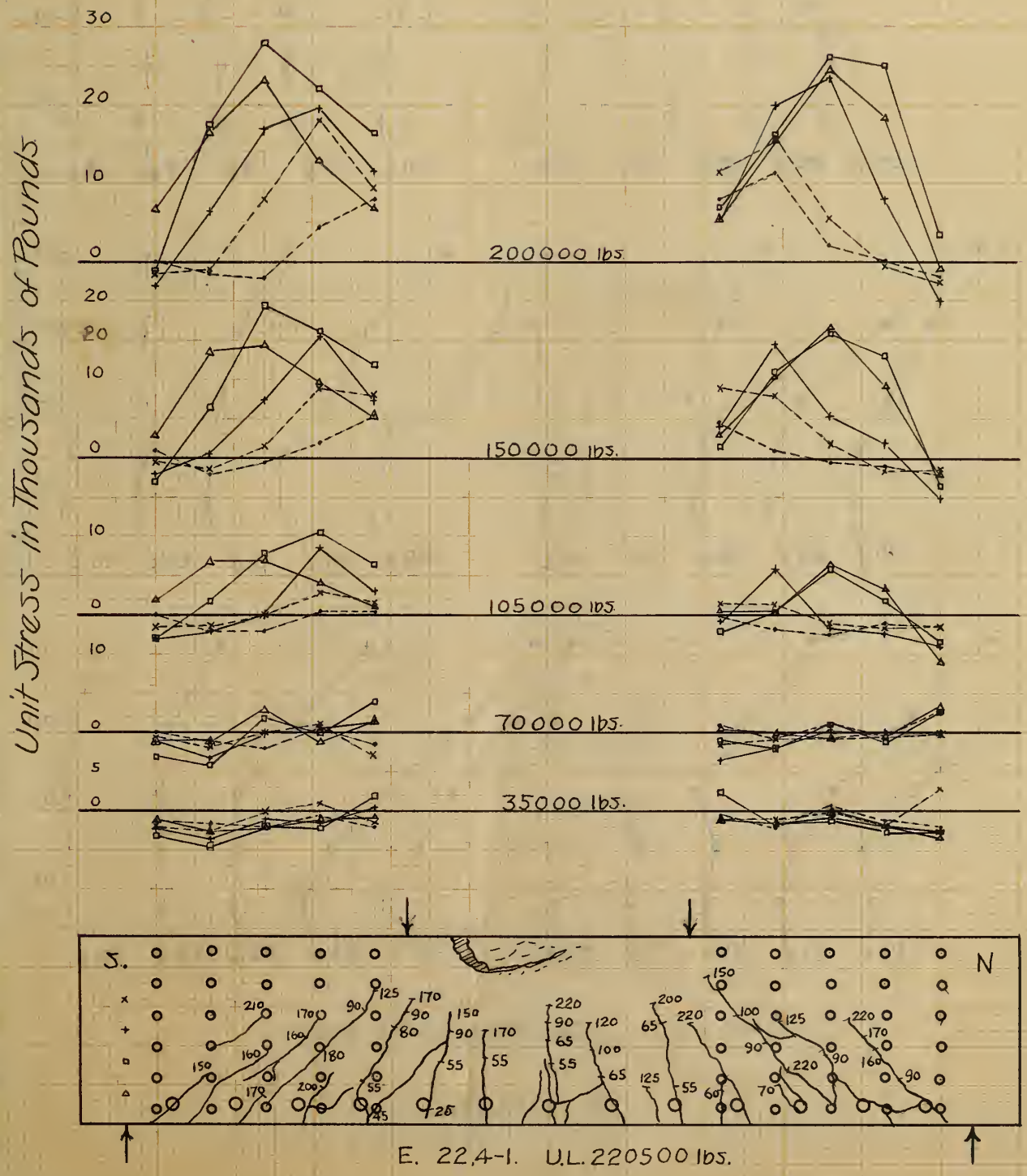
22.4-1 W.

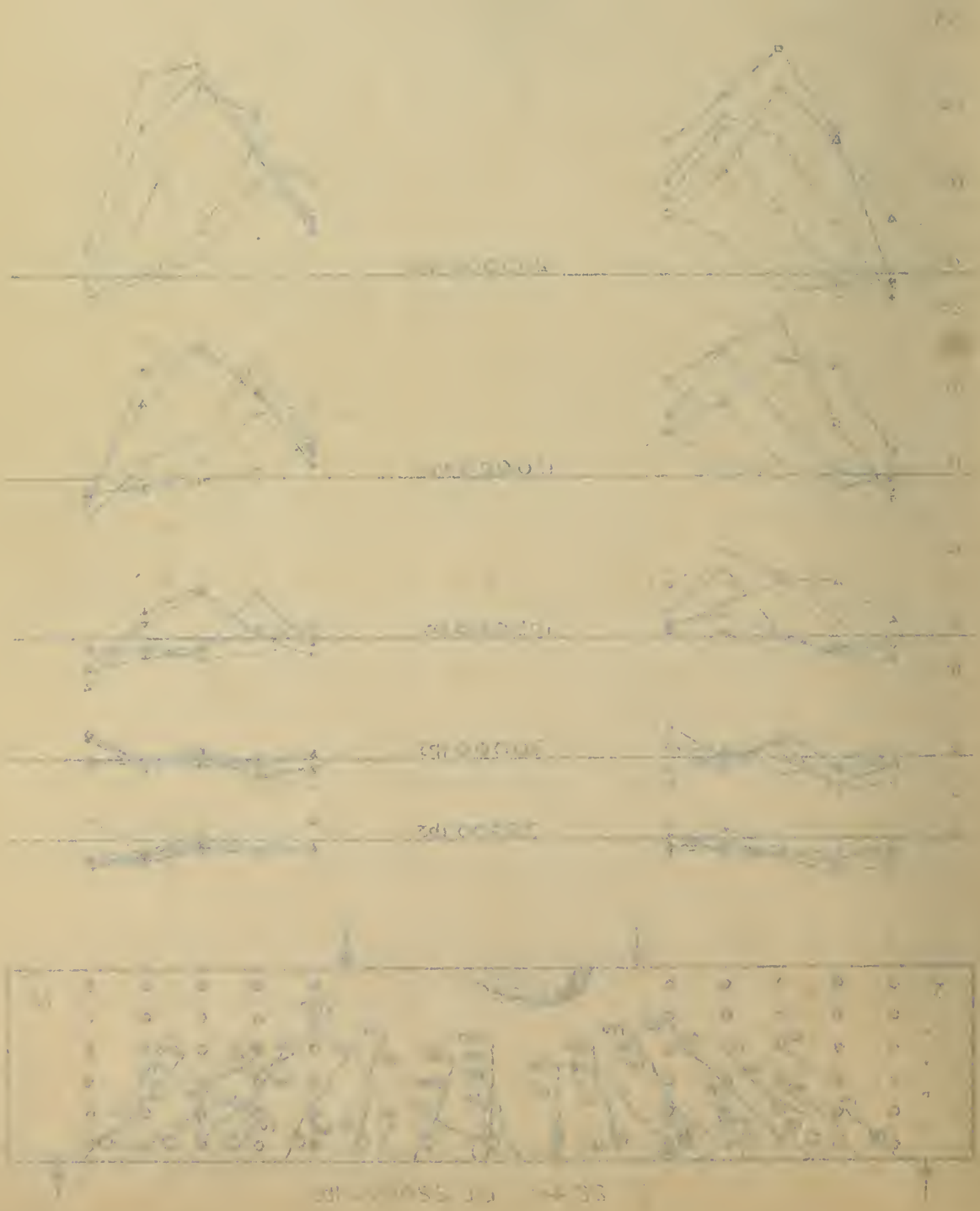


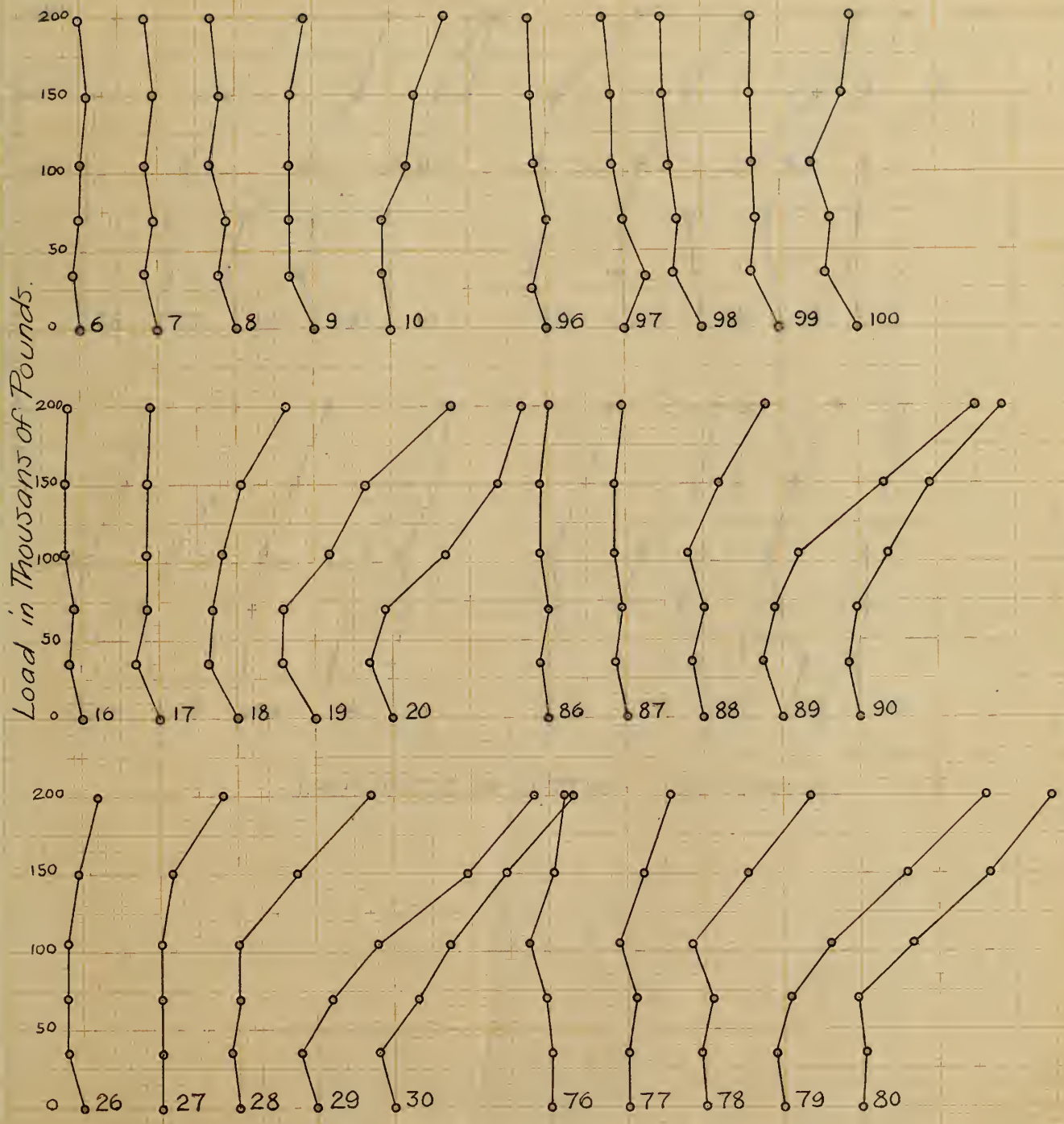
Unit Stress in Stirrups. 1 in. = 20000 lbs.



E. 224-1. U.L. 220500 lbs.


Unit Stress in Thousands of Pounds
30
20
10
0
200000 lbs.
20
20
10
0
150000 lbs.
10
0
105000 lbs.
10
0
70000 lbs.
5
0
35000 lbs.
S.
N
E. 22.4-1. U.L. 220500 lbs.

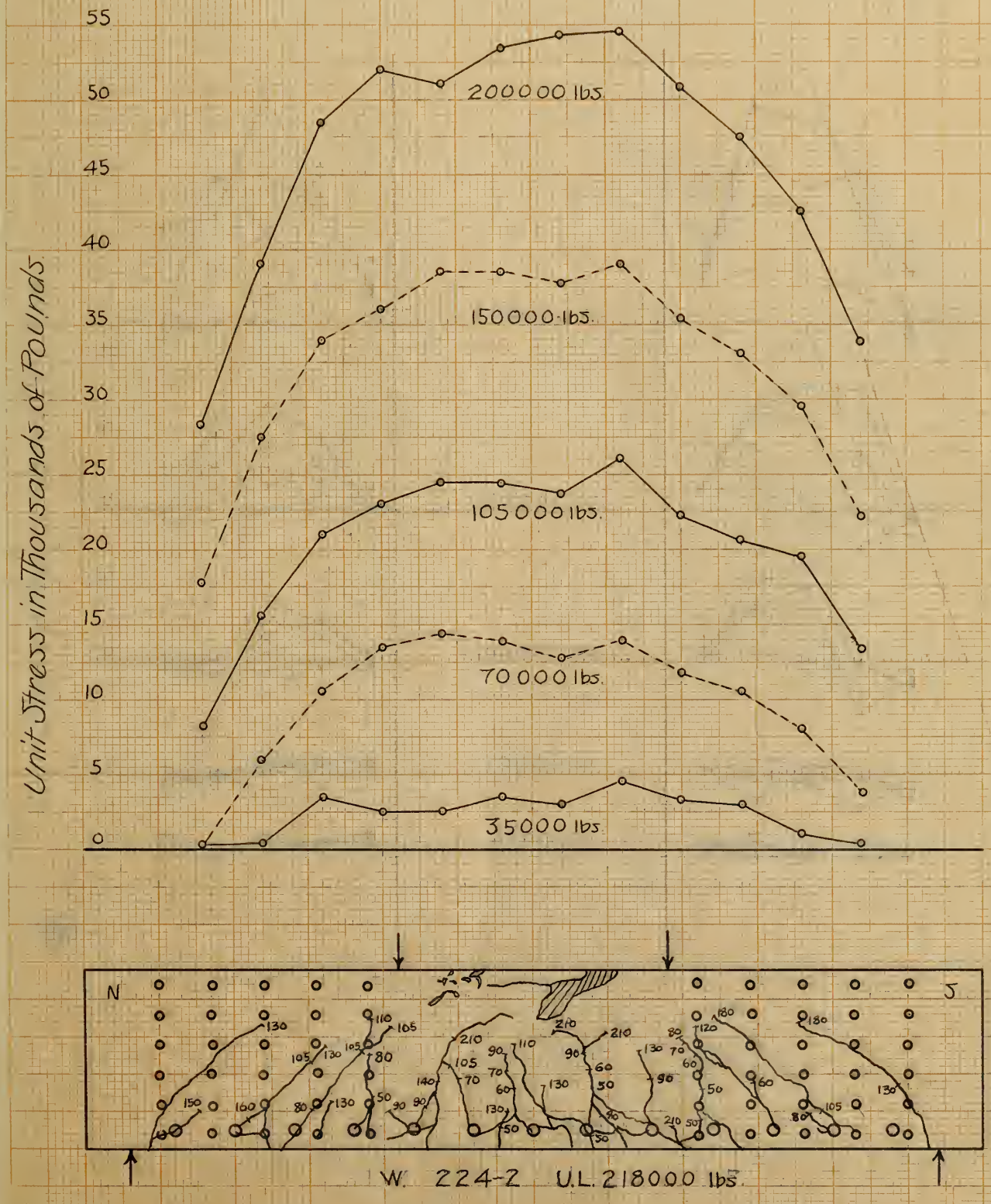


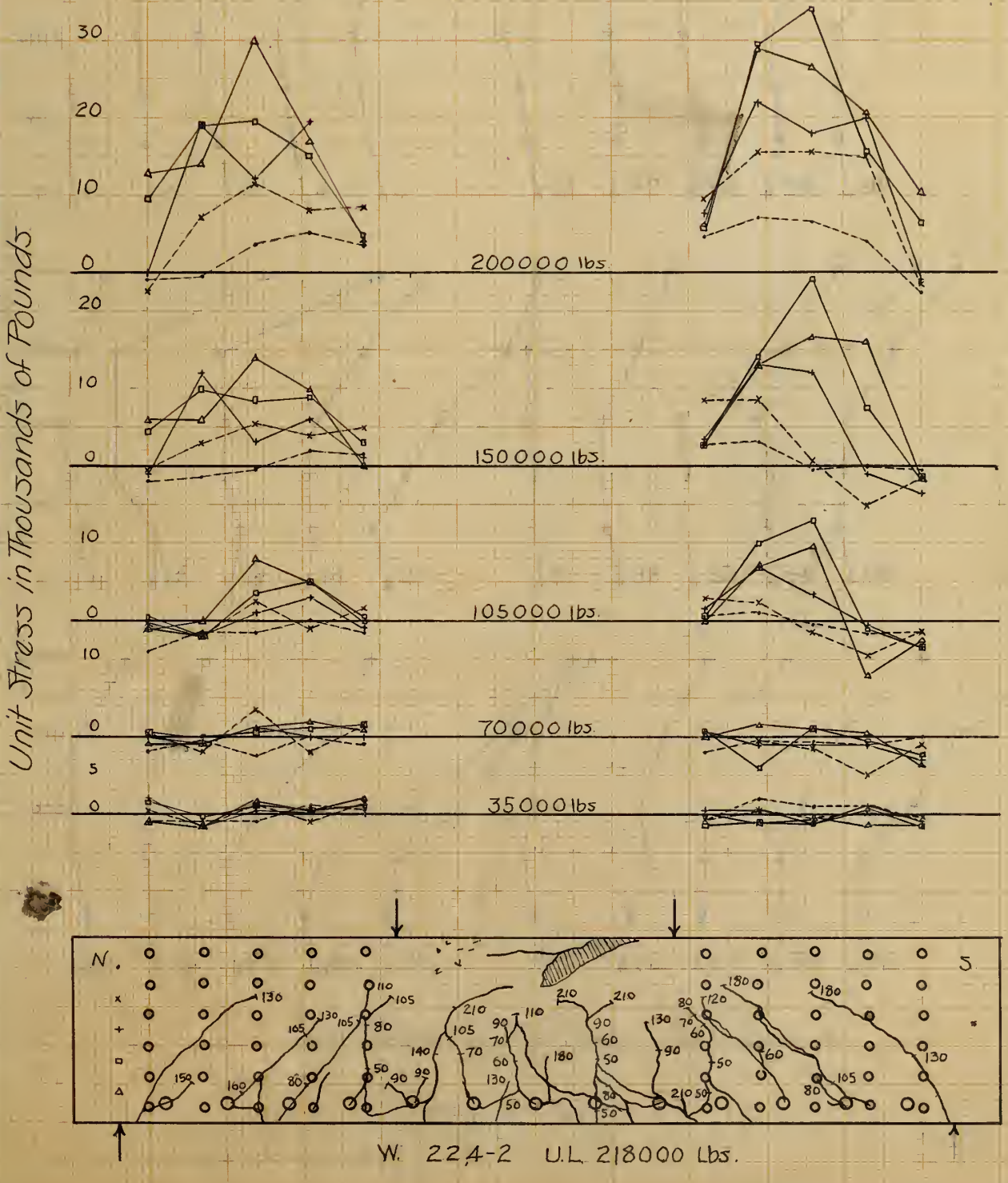


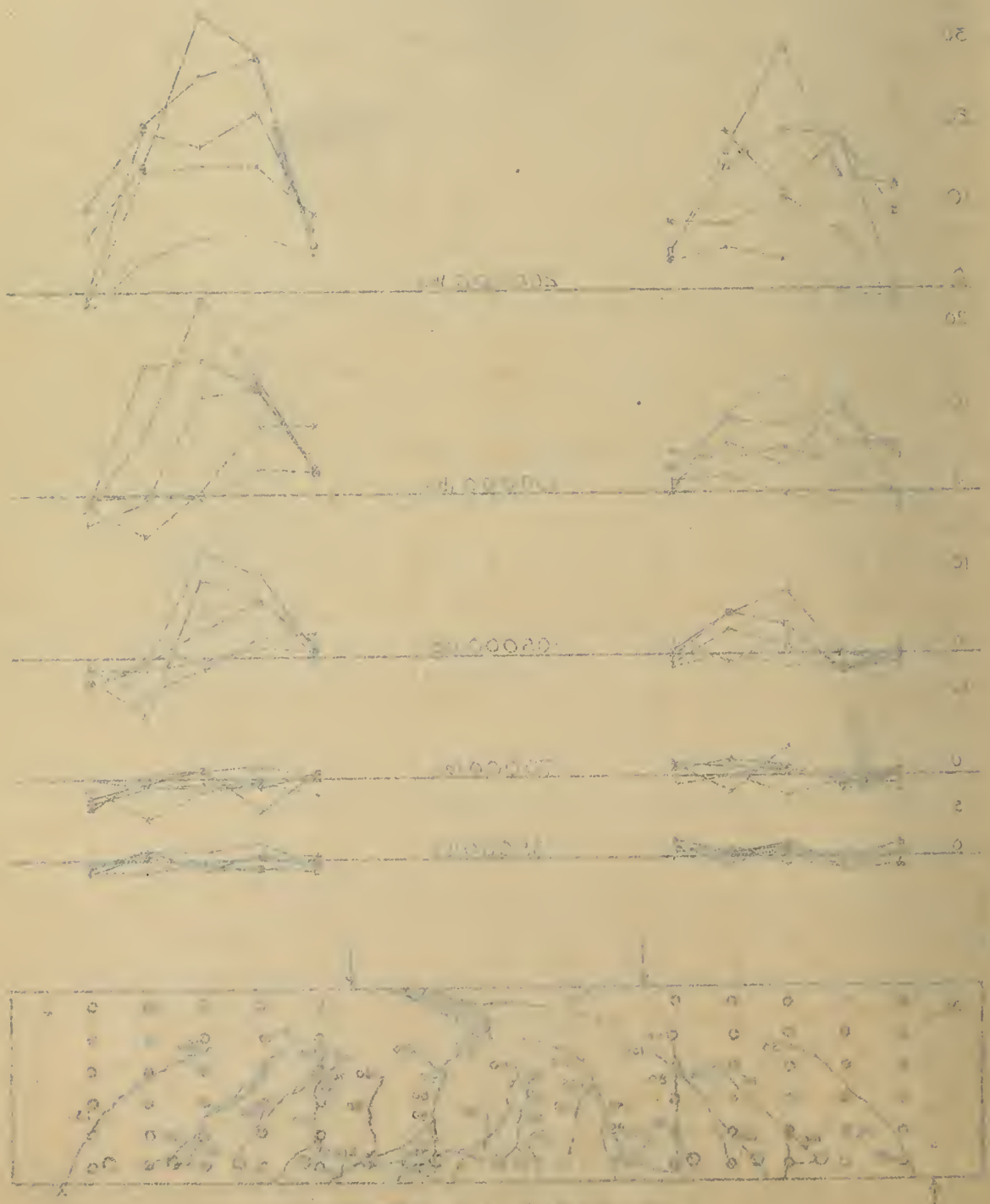
Unit Stress in Stirrups 1 in = 20000 lbs.

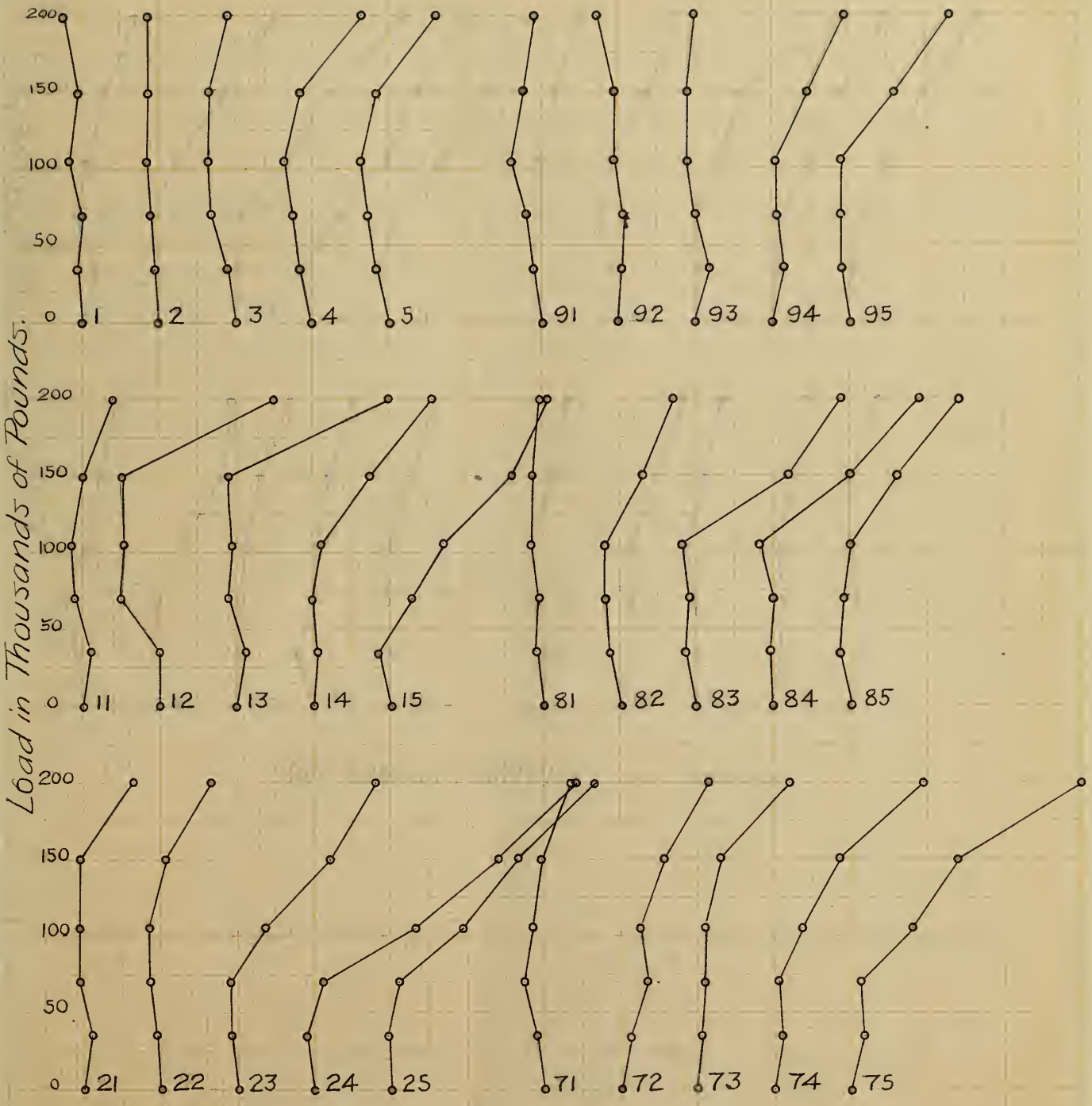
22,4-1 E.

Unit Stress in Stirrups 1 in. = 20 000 lbs.


Unit Stress in Thousands of Pounds
55
50
45
40
35
30
25
20
15
10
5
0
200000 lbs.
150000 lbs.
105000 lbs.
70000 lbs.
35000 lbs.
N
S
W. 224-2 U.L. 218000 lbs.

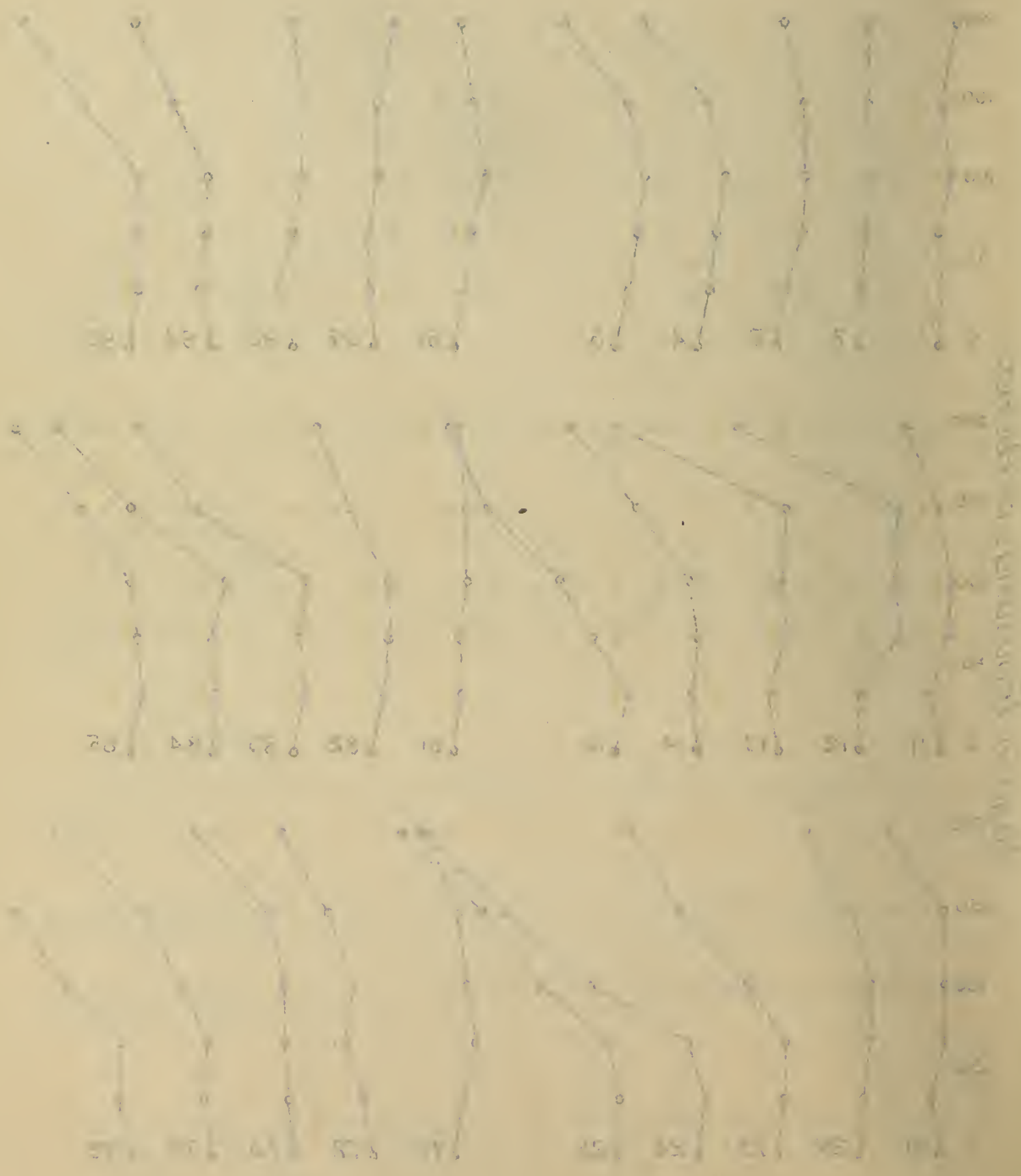

Unit Stress in Thousands of Pounds
30
20
10
0
200000 lbs.
20
10
0
150000 lbs.
10
0
10
105000 lbs.
0
5
70000 lbs.
0
35000 lbs
N.
S
W. 22,4-2 U.L. 218000 Lbs.

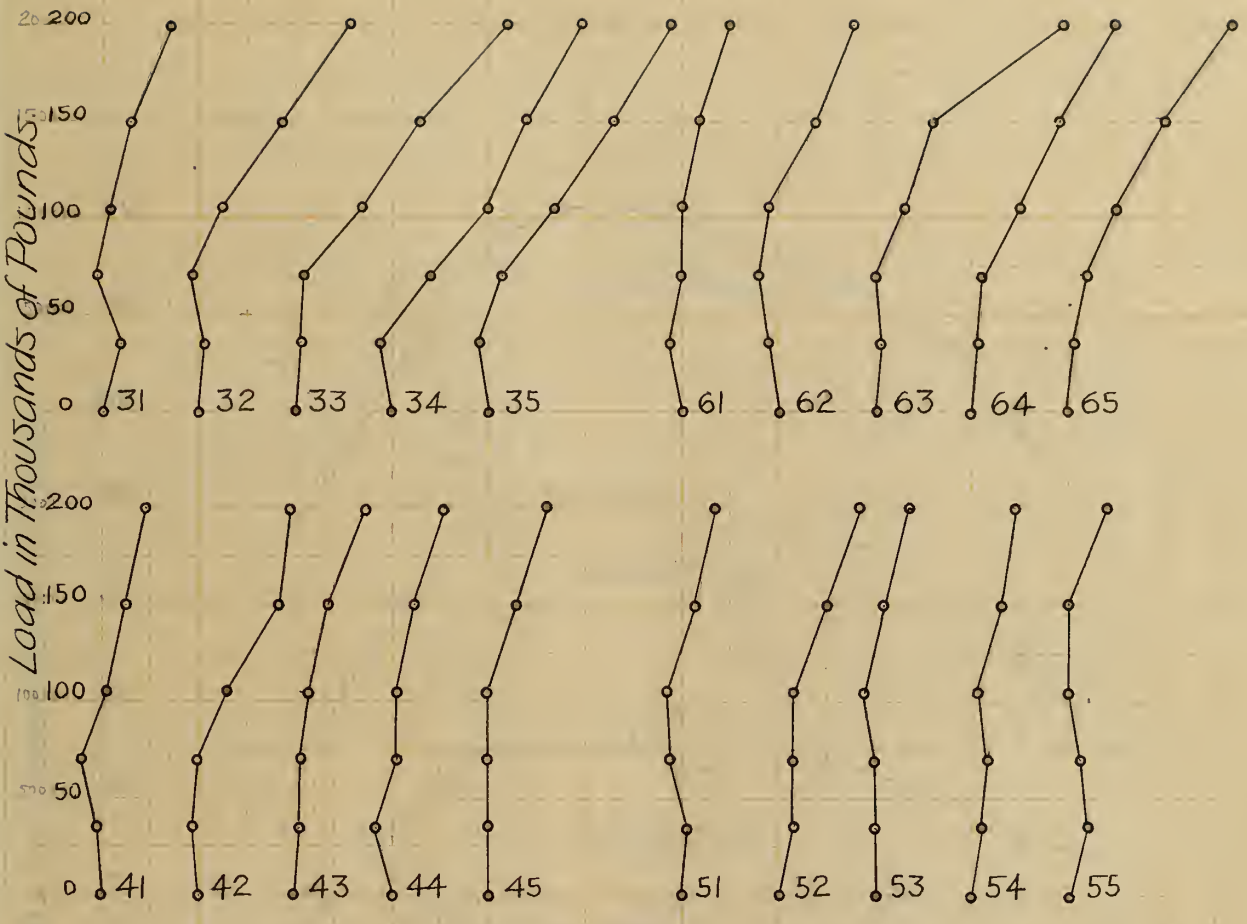




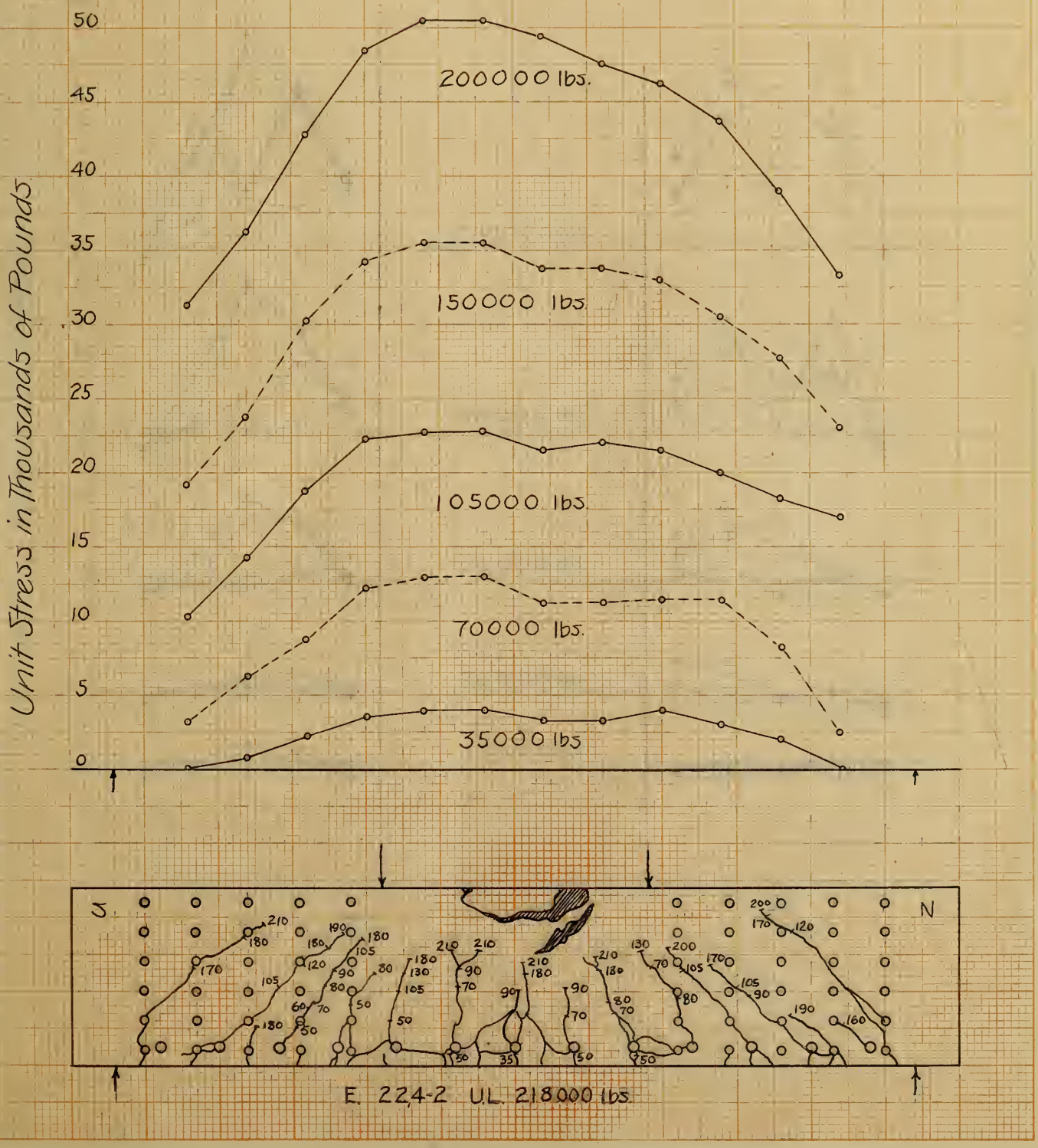
Unit Stress in Stirrups 1 in = 20000 lbs.

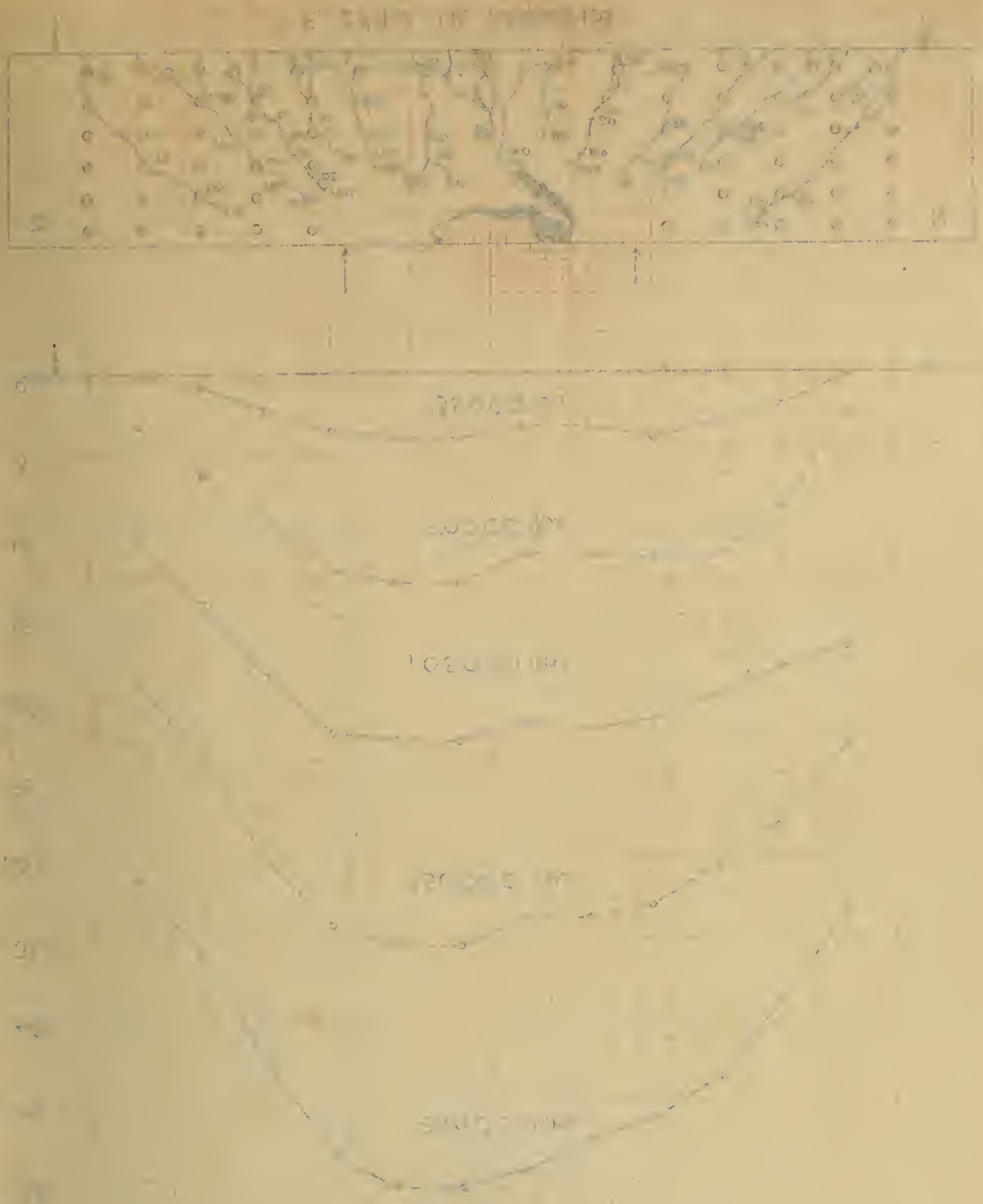
224-2 W.N.

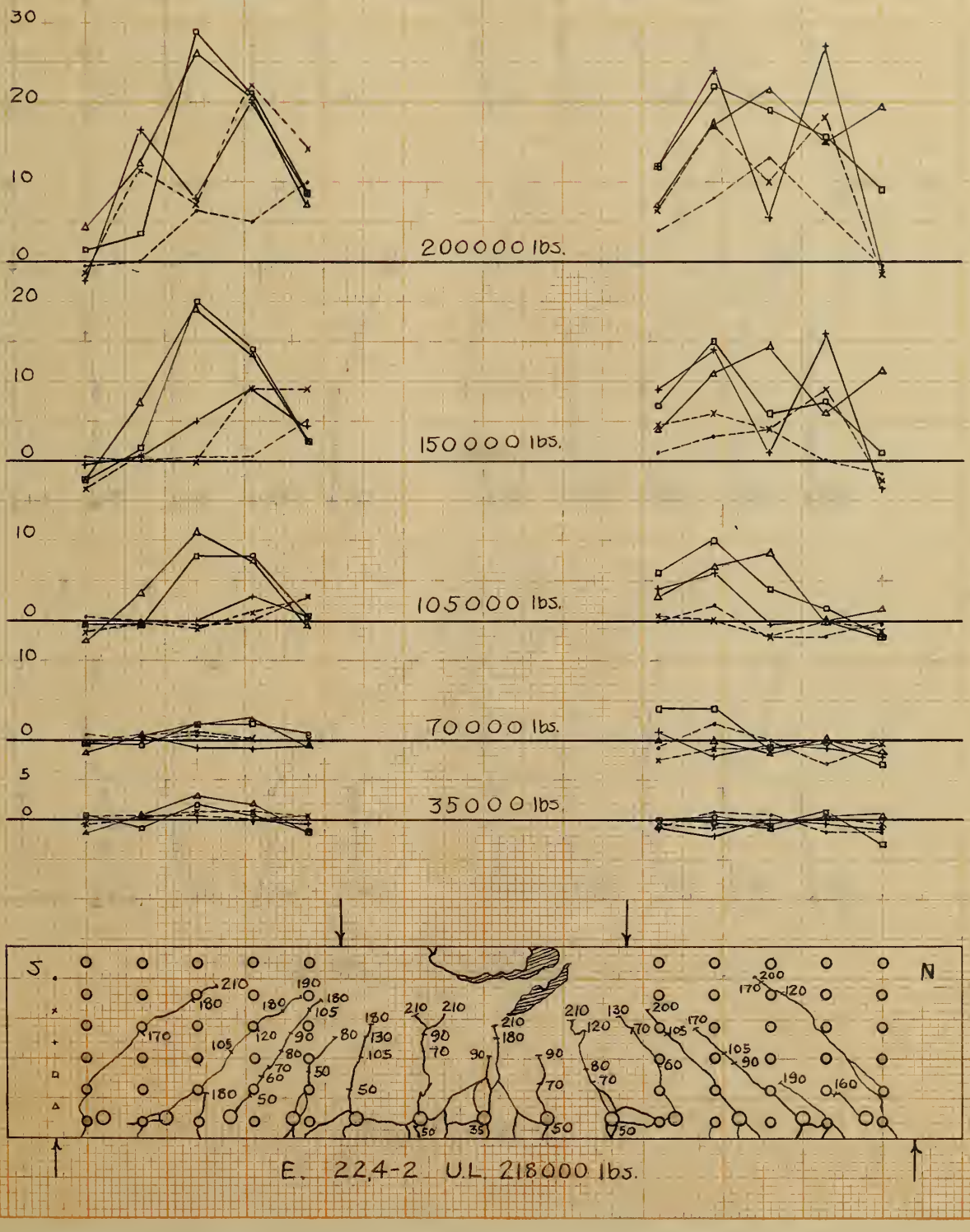


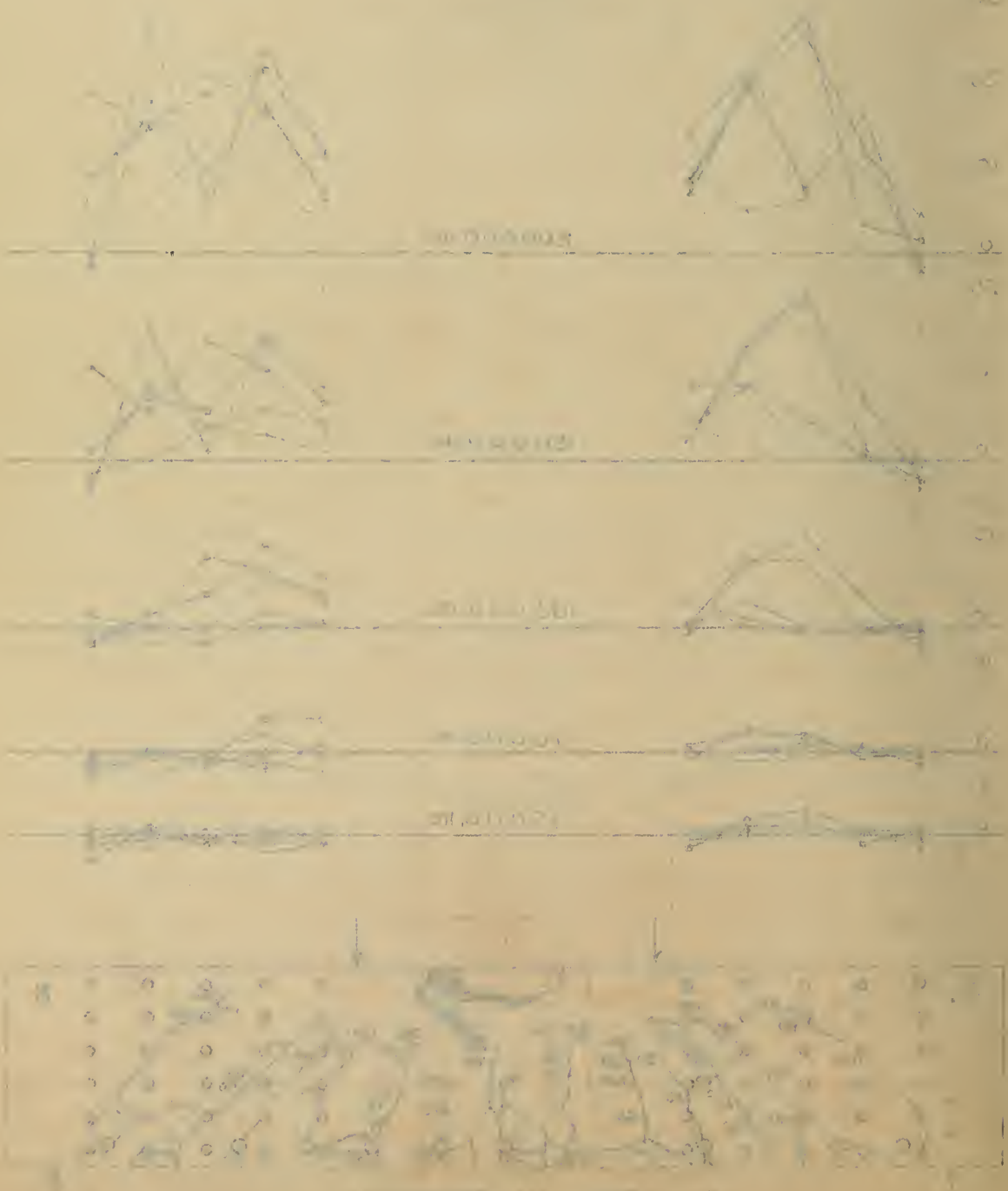


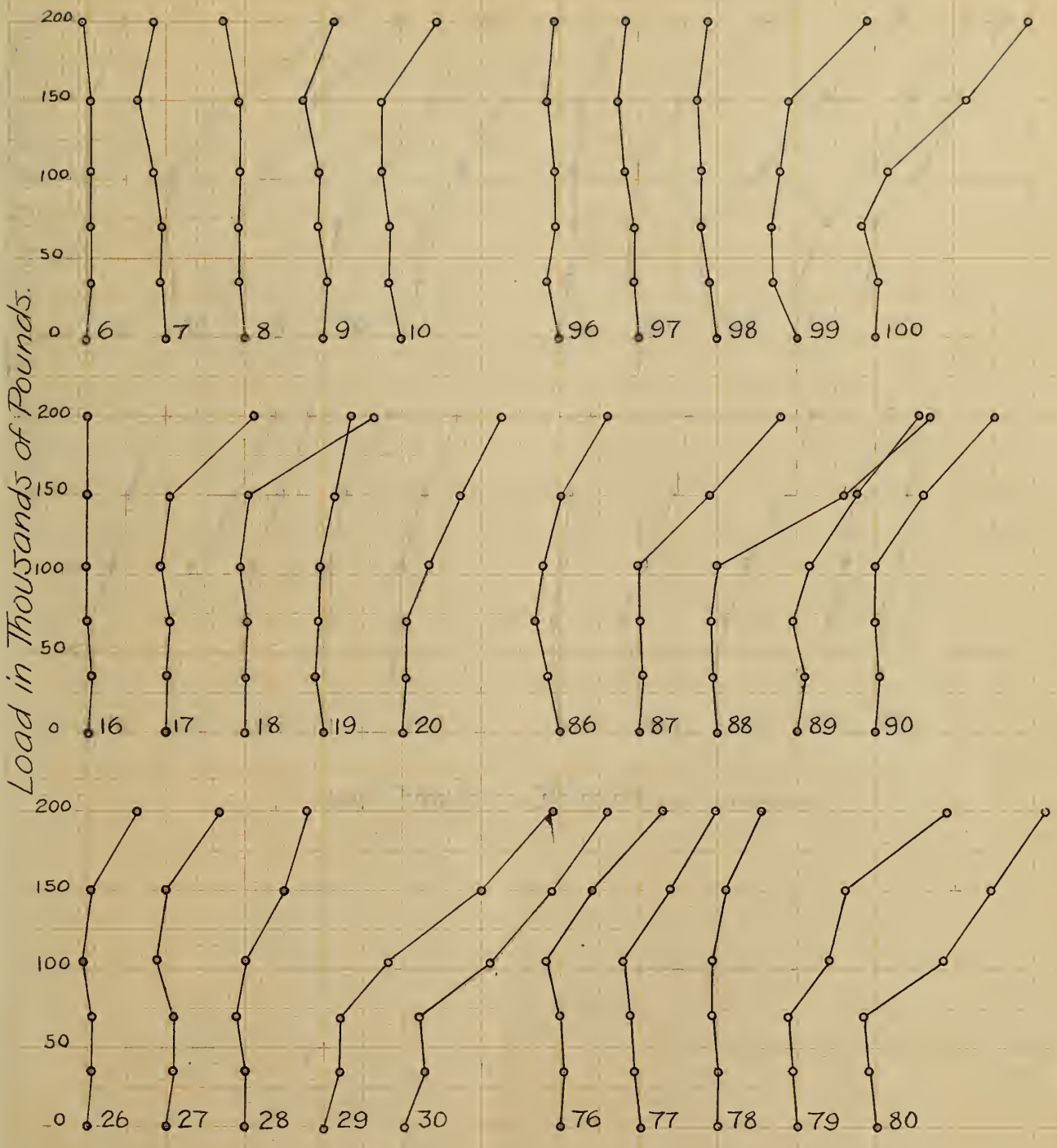
Unit Stress in Stirrups. 1 in. = 20000 lbs.


Unit Stress in Thousands of Pounds
50
45
40
35
30
25
20
15
10
5
0
200000 lbs.
150000 lbs.
105000 lbs.
70000 lbs.
35000 lbs
S
N
E. 224-2 U.L. 218000 lbs.

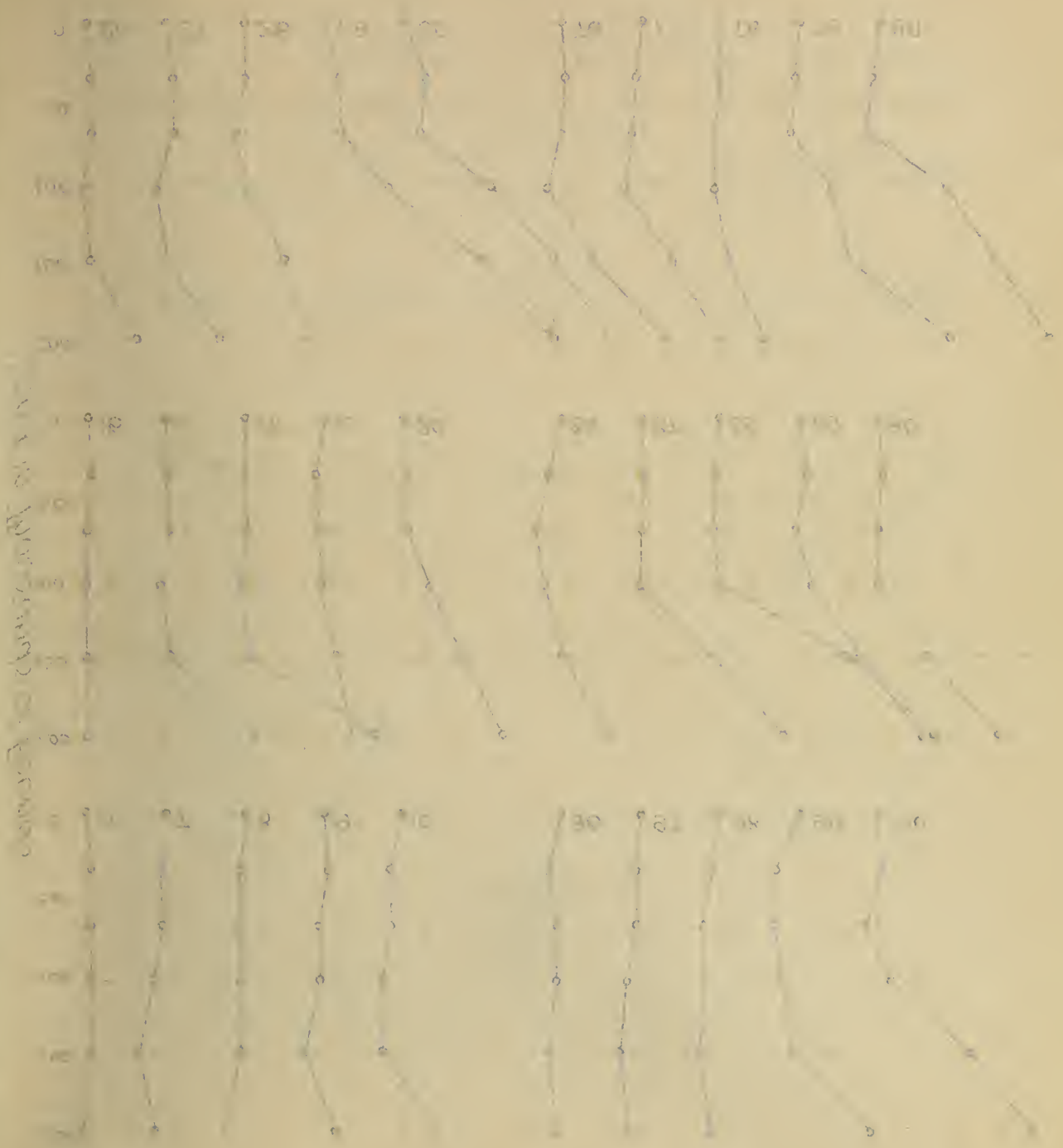


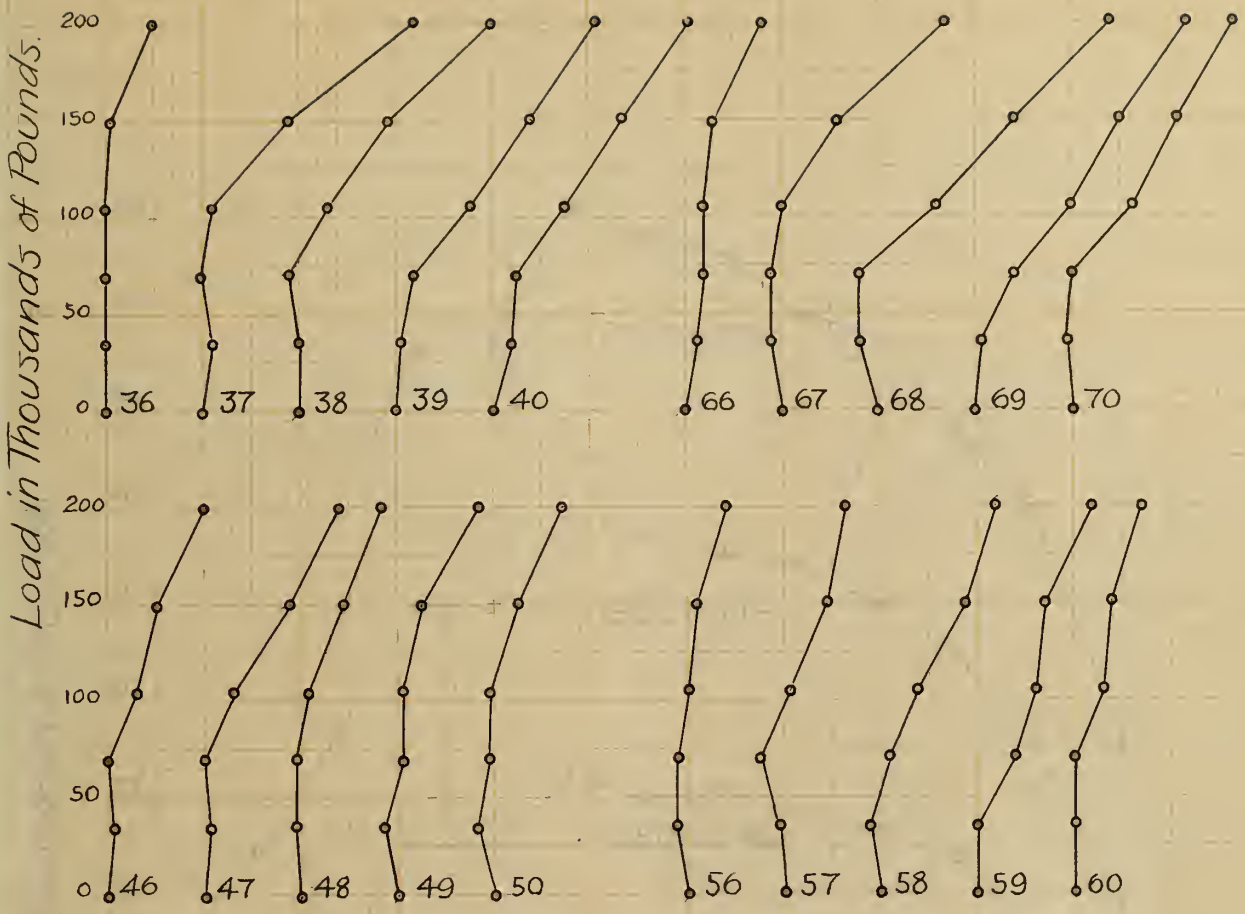

Unit Stress in Thousands of Pounds
30
20
10
0
200000 lbs.
20
10
0
150000 lbs.
10
0
105000 lbs.
10
0
70000 lbs.
5
0
35000 lbs.
S
N
E. 22,4-2 U.L. 218000 lbs.




Load in Thousands of Pounds.
200
150
100
50
0
6
7
8
9
10
96
97
98
99
100
16
17
18
19
20
86
87
88
89
90
26
27
28
29
30
76
77
78
79
80
Unit Stress in Stirrups 1/in = 20 000 lbs.

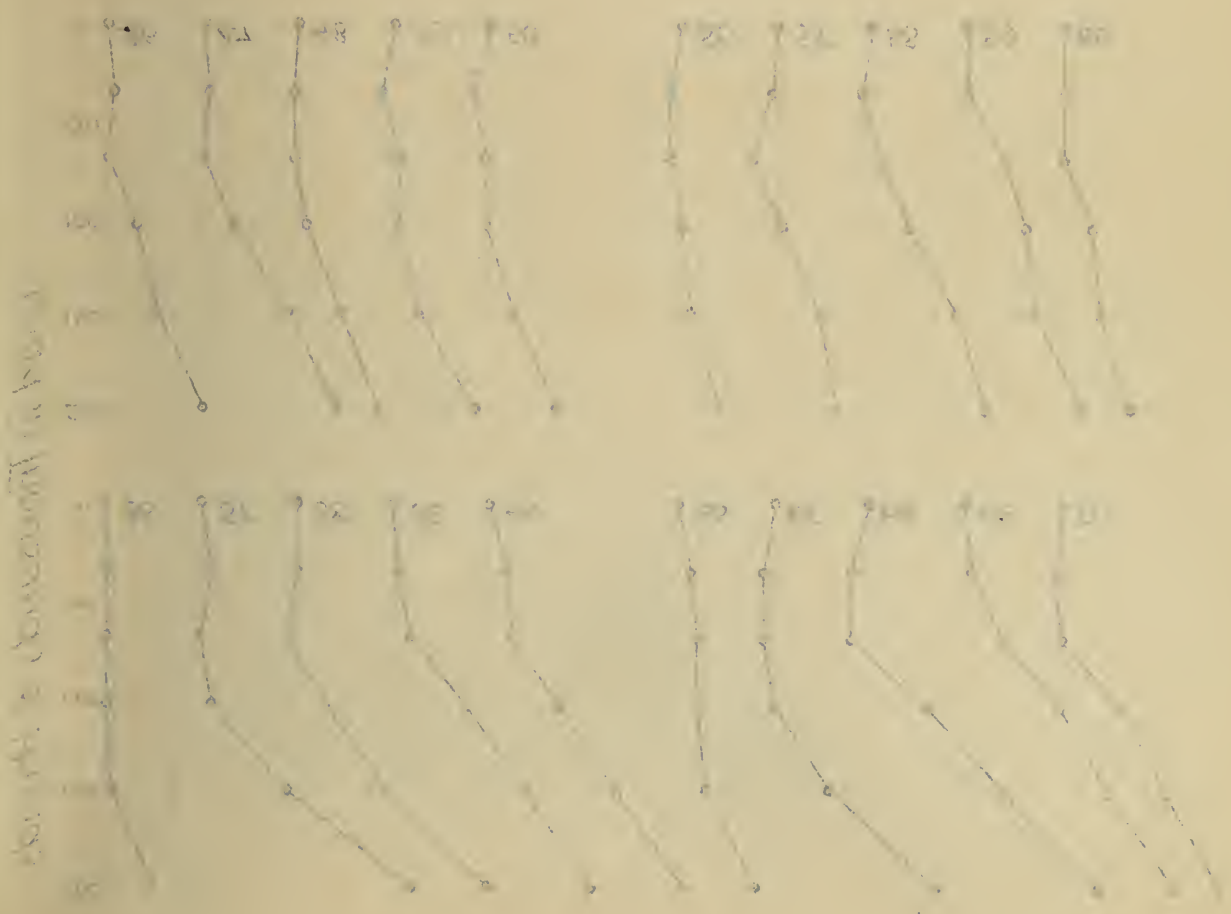
22-4-2. E.

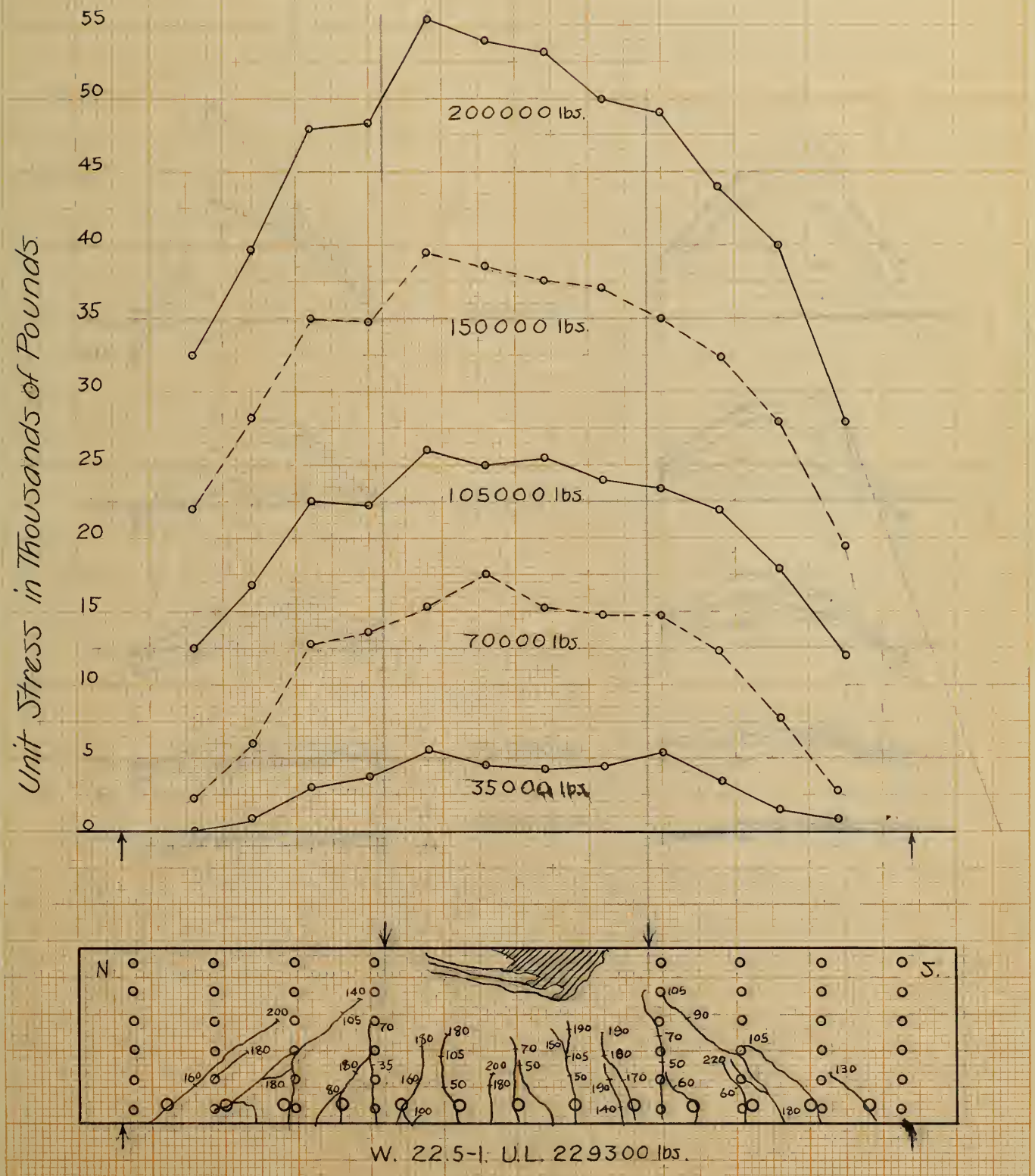


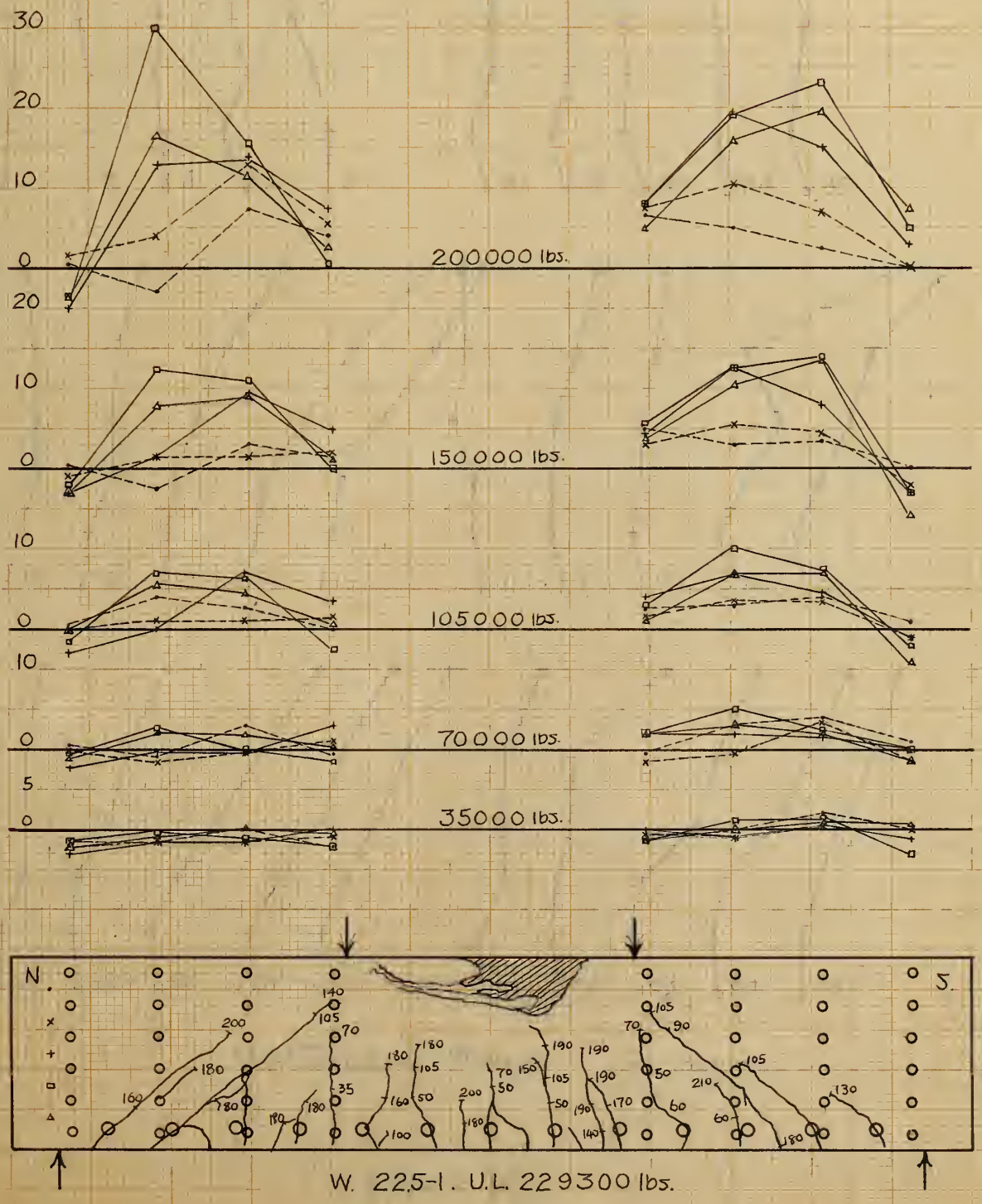


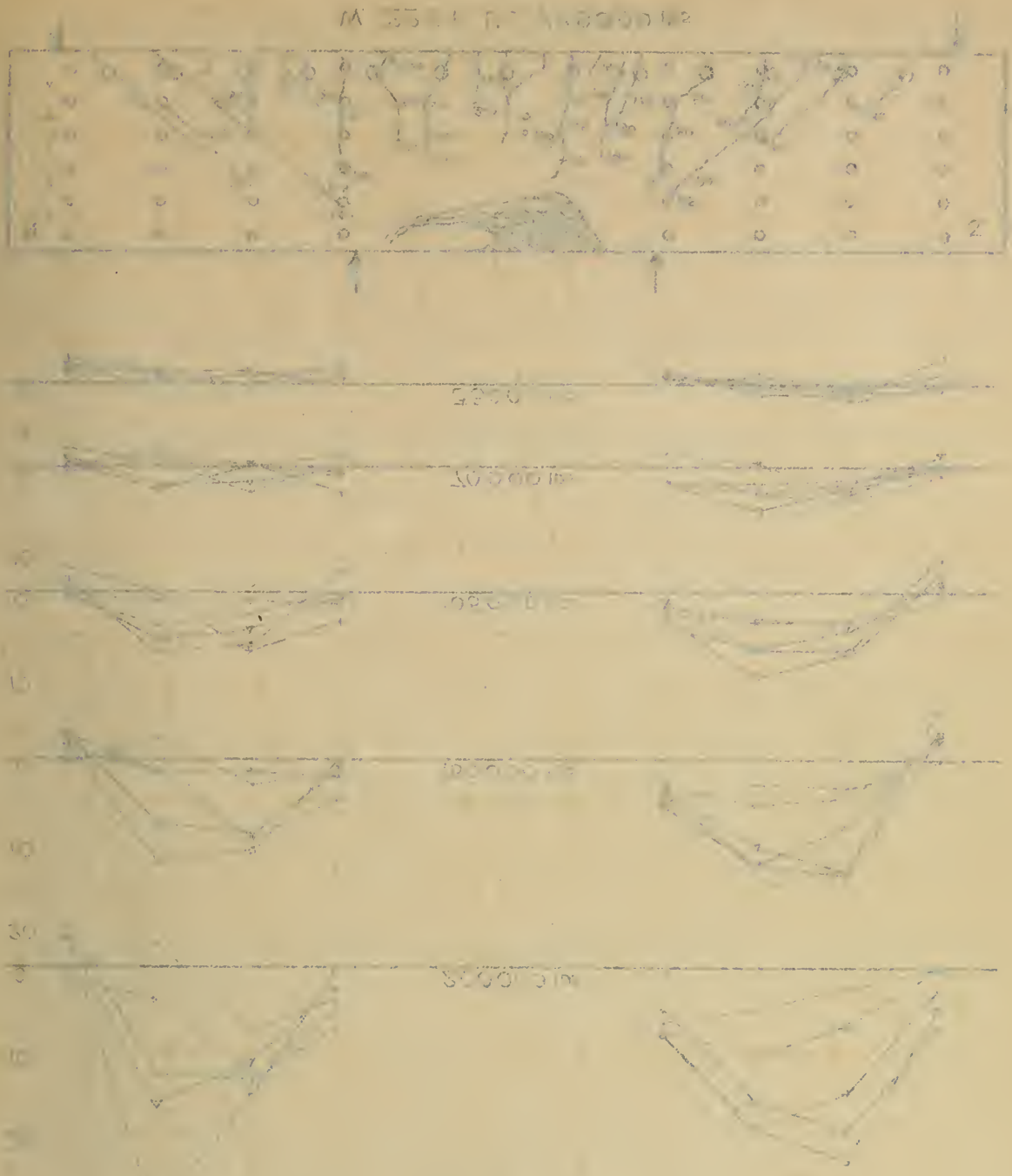
Unit Stress in Stirrups. 1 in = 20000 lbs.

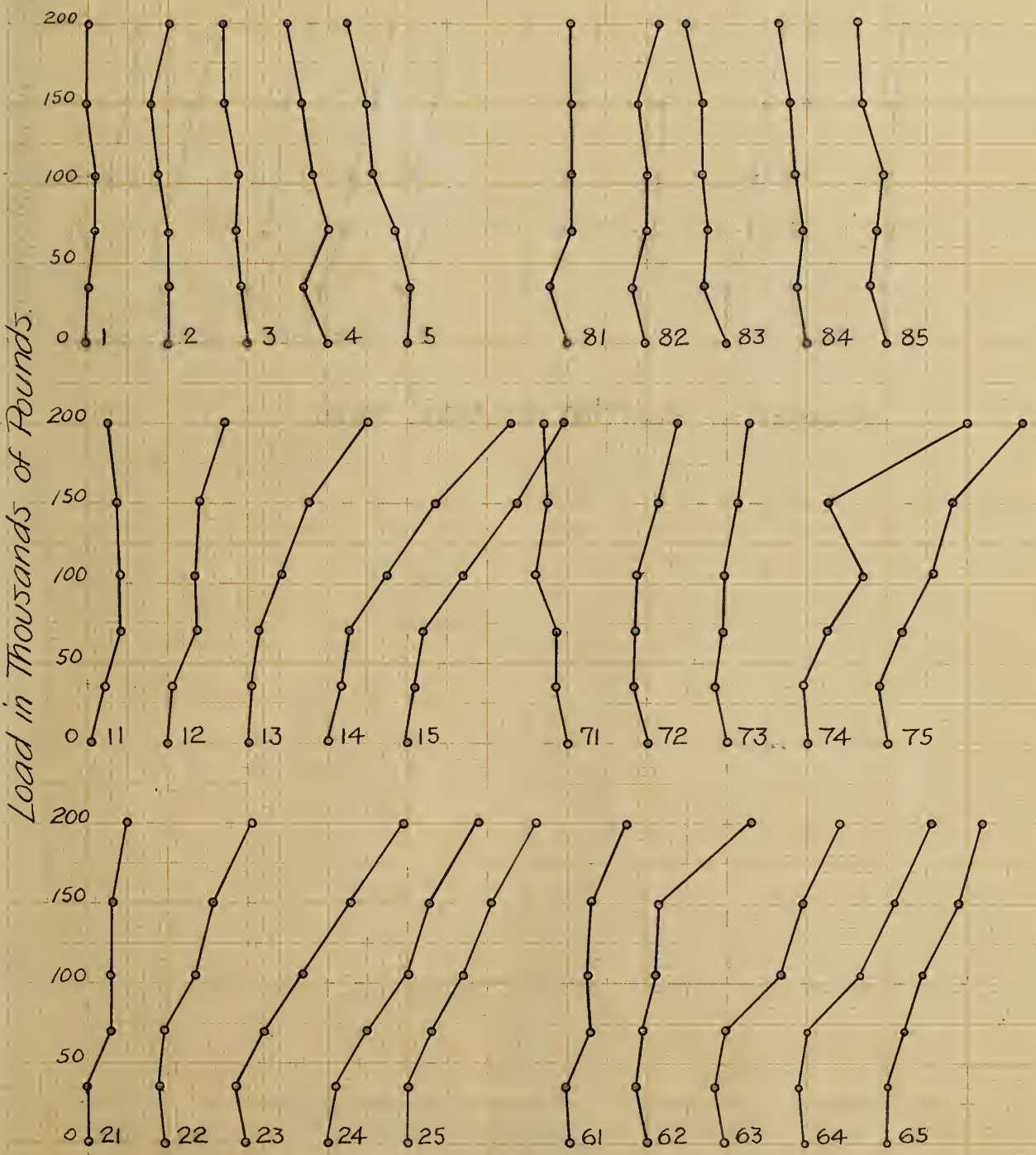
22.4-2 E.




Unit Stress in Thousands of Pounds
55
50
45
40
35
30
25
20
15
10
5
0
200000 lbs.
150000 lbs.
105000 lbs.
70000 lbs.
35000 lbs.
N.
S.
W. 22.5-1. U.L. 229300 lbs.

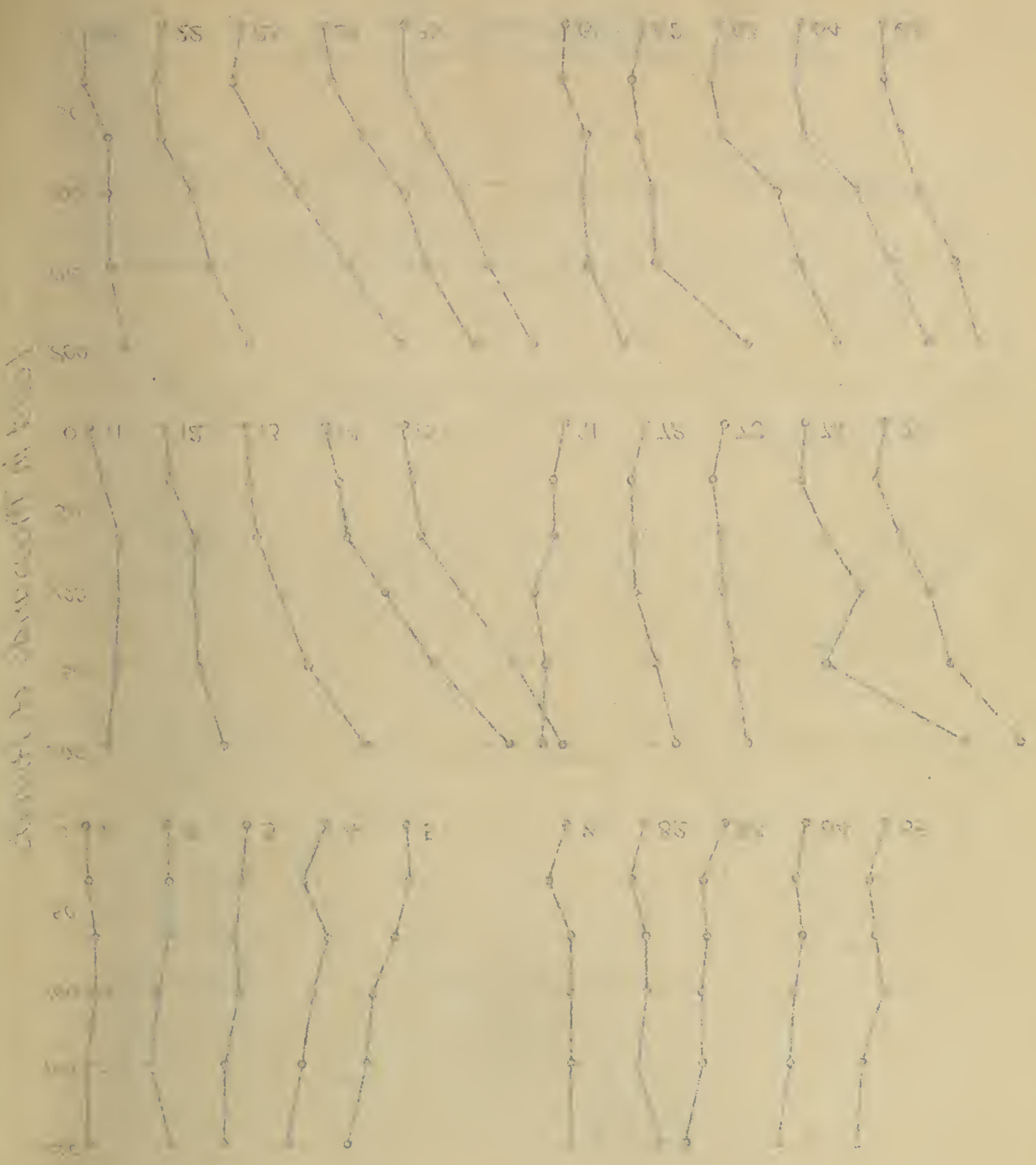

Unit Stress in Thousands of Pounds
30
20
10
0
200000 lbs.
20
10
0
150000 lbs.
10
0
105000 lbs.
10
0
70000 lbs.
5
0
35000 lbs.
N.
S.
W. 22.5-1. U.L. 229300 lbs.

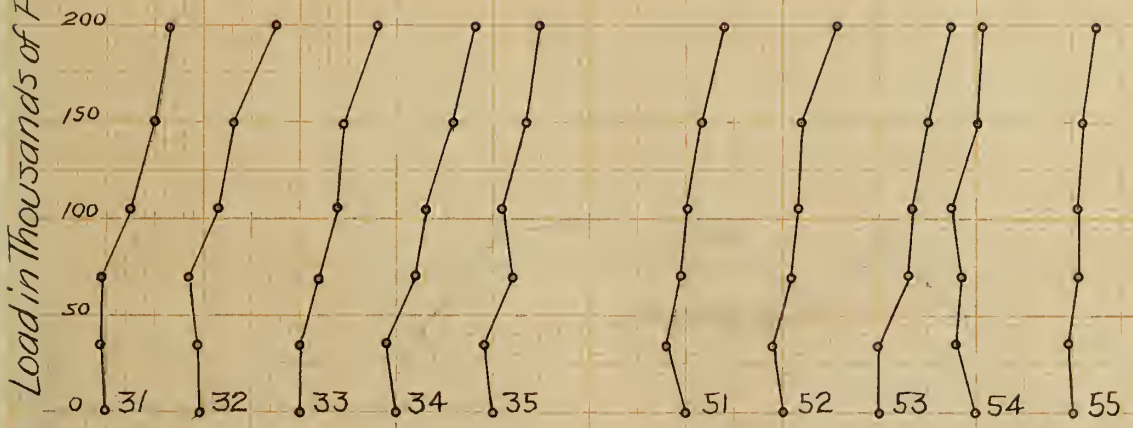




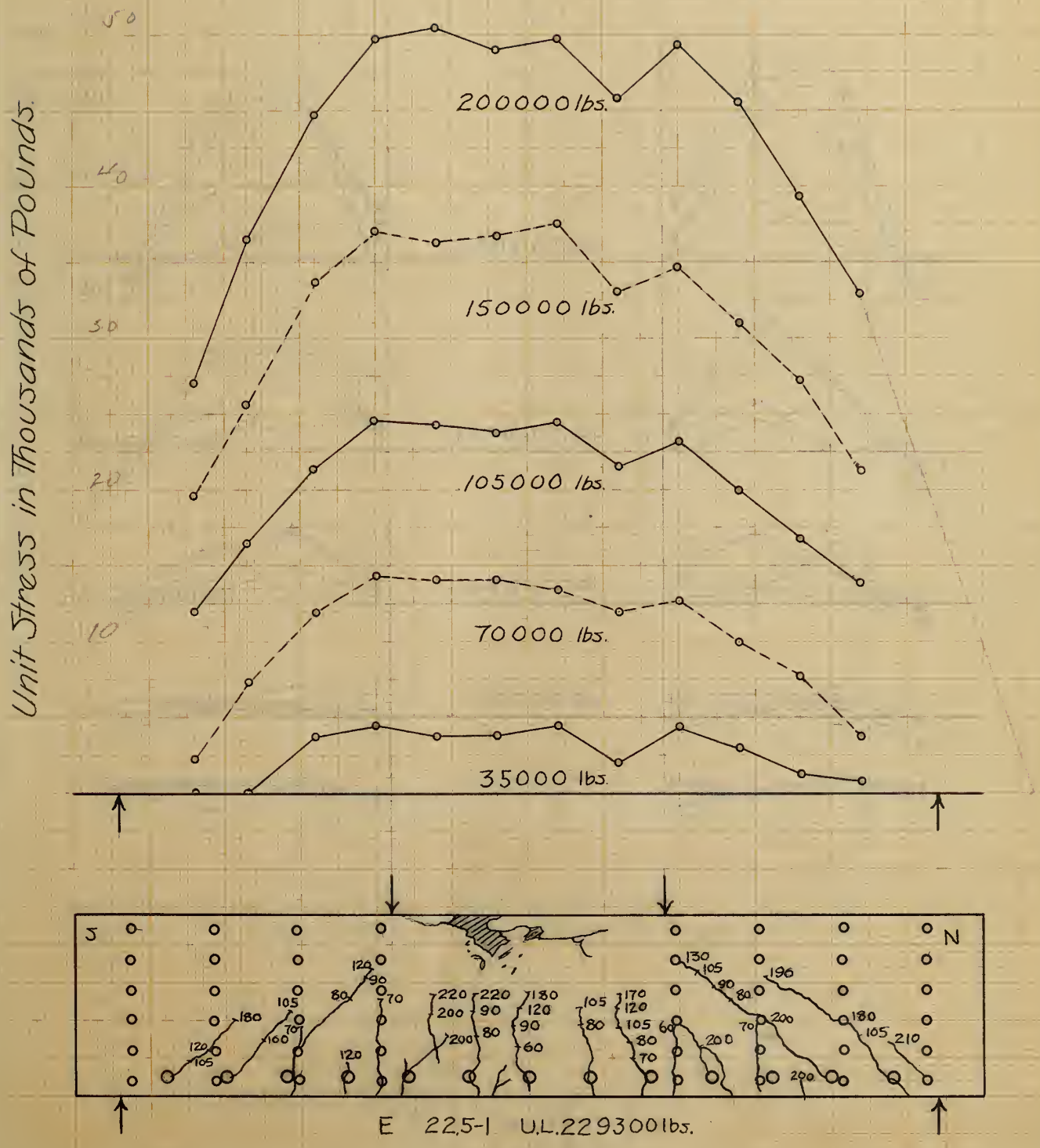
Unit Stress in Stirrups 1 in. = 20000 lbs.

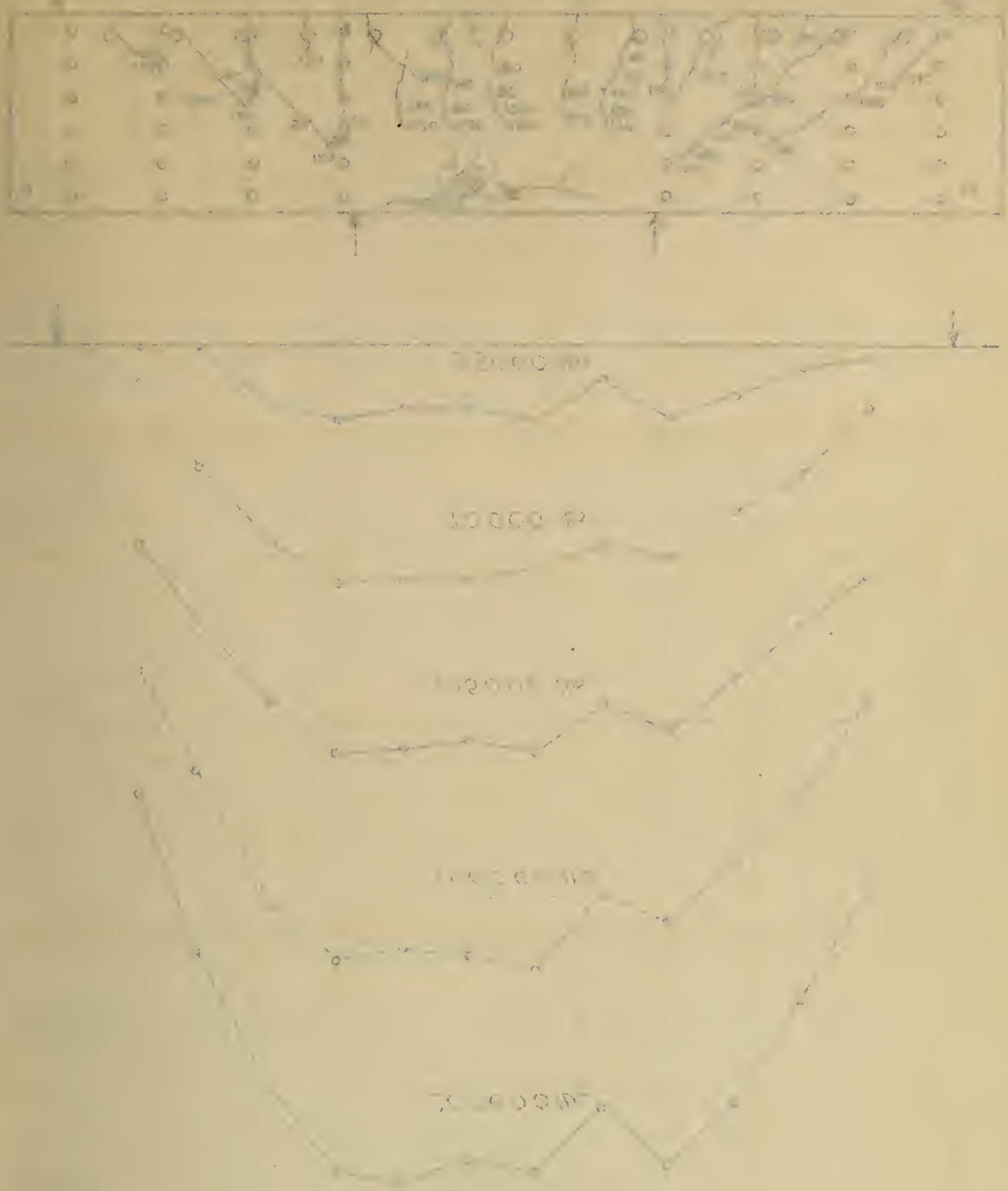
22.5-1 W.

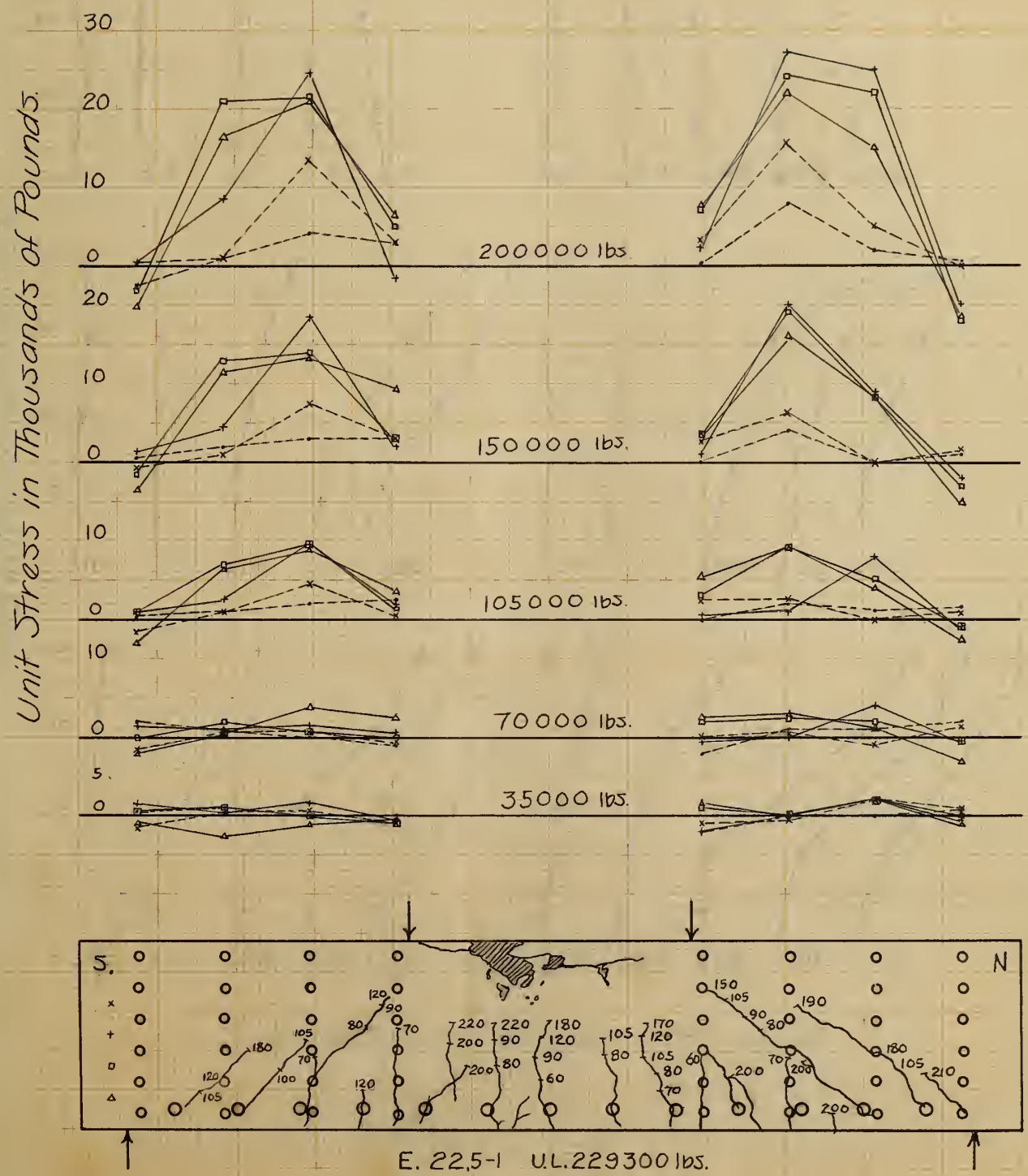


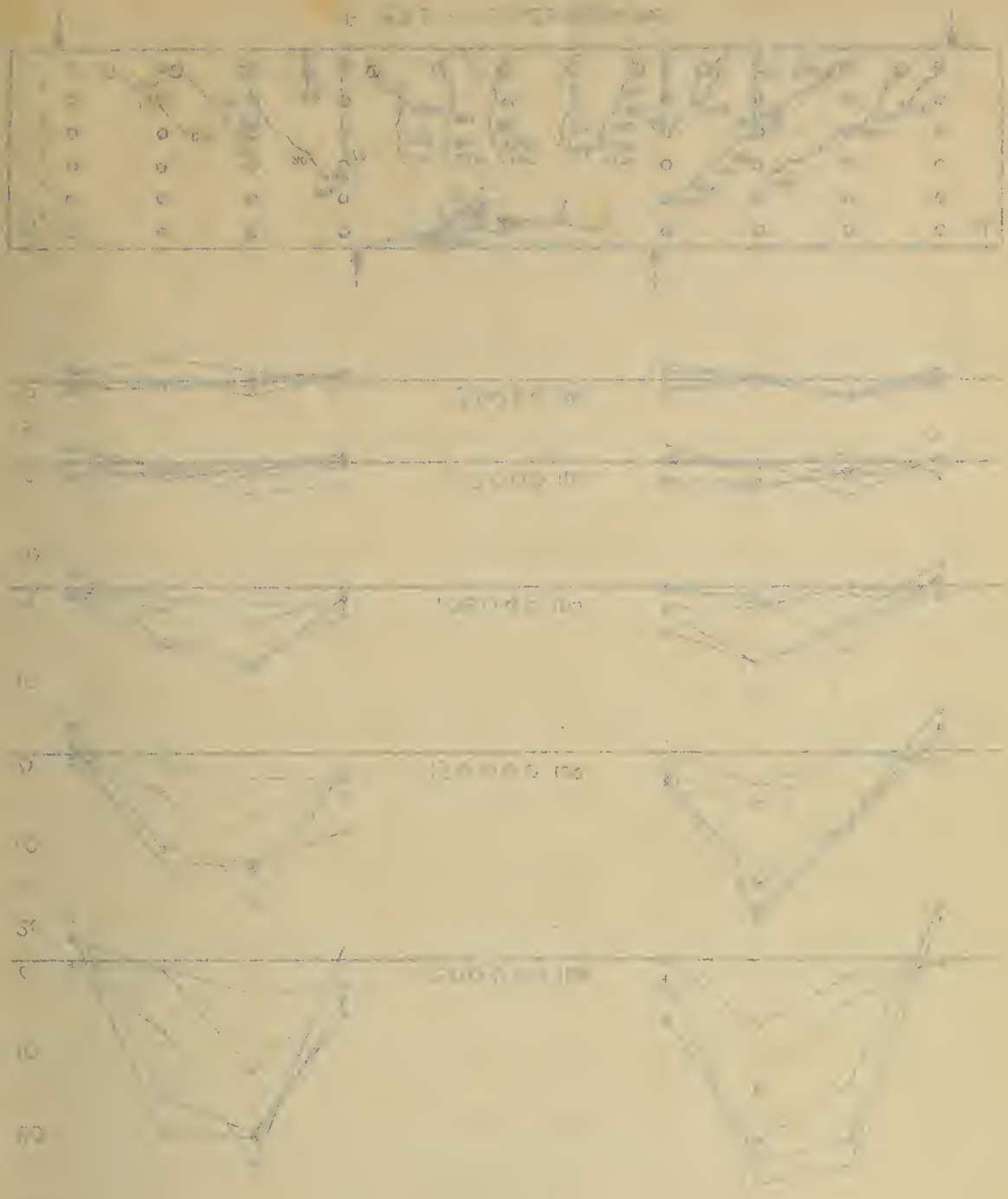


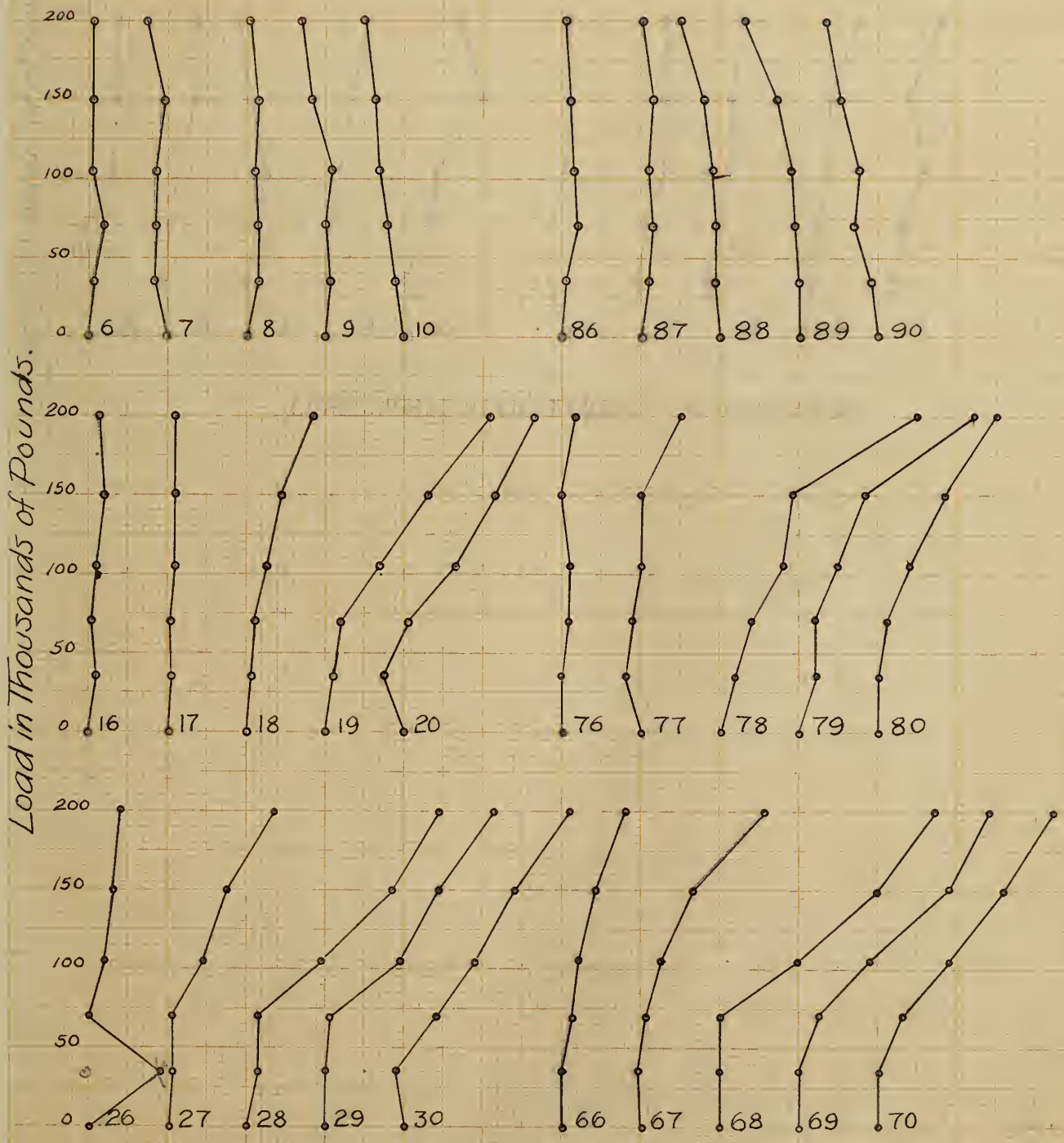
Unit Stress in Stirrups. 1/n = 20000/65.


Unit Stress in Thousands of Pounds.
50
40
30
20
10
200000 lbs.
150000 lbs.
105000 lbs.
70000 lbs.
35000 lbs.
S
N
E 22.5-1 U.L. 229300 lbs.



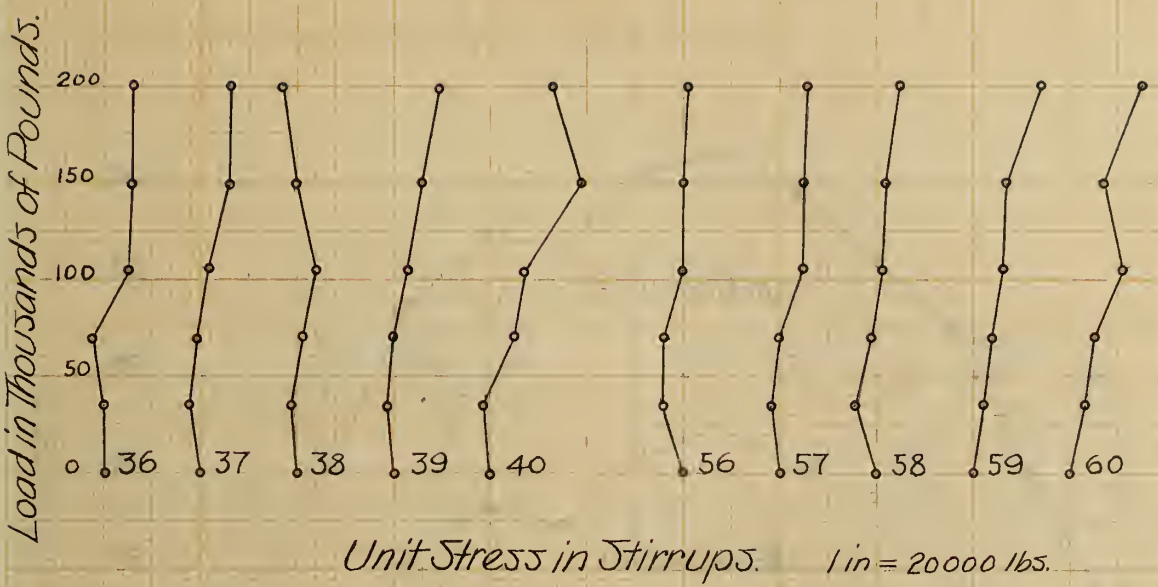

Unit Stress in Thousands of Pounds.
30
20
10
0
200000 lbs.
20
10
0
150000 lbs.
10
0
105000 lbs.
10
0
70000 lbs.
5.
0
35000 lbs.
5.
N
E. 22,5-1 U.L. 229300 lbs.





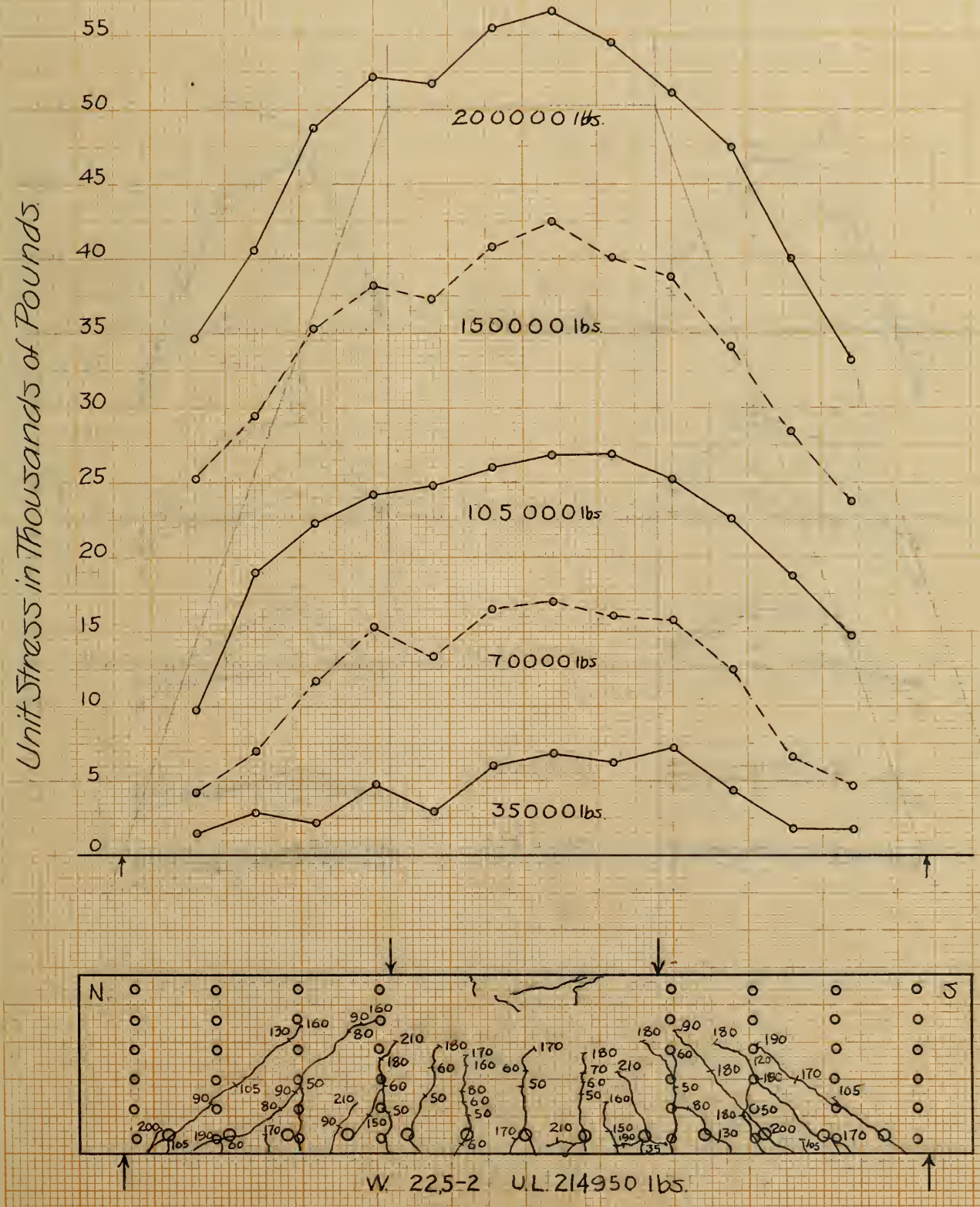
Unit Stress in Stirrups. 1 in. = 20000 lbs.

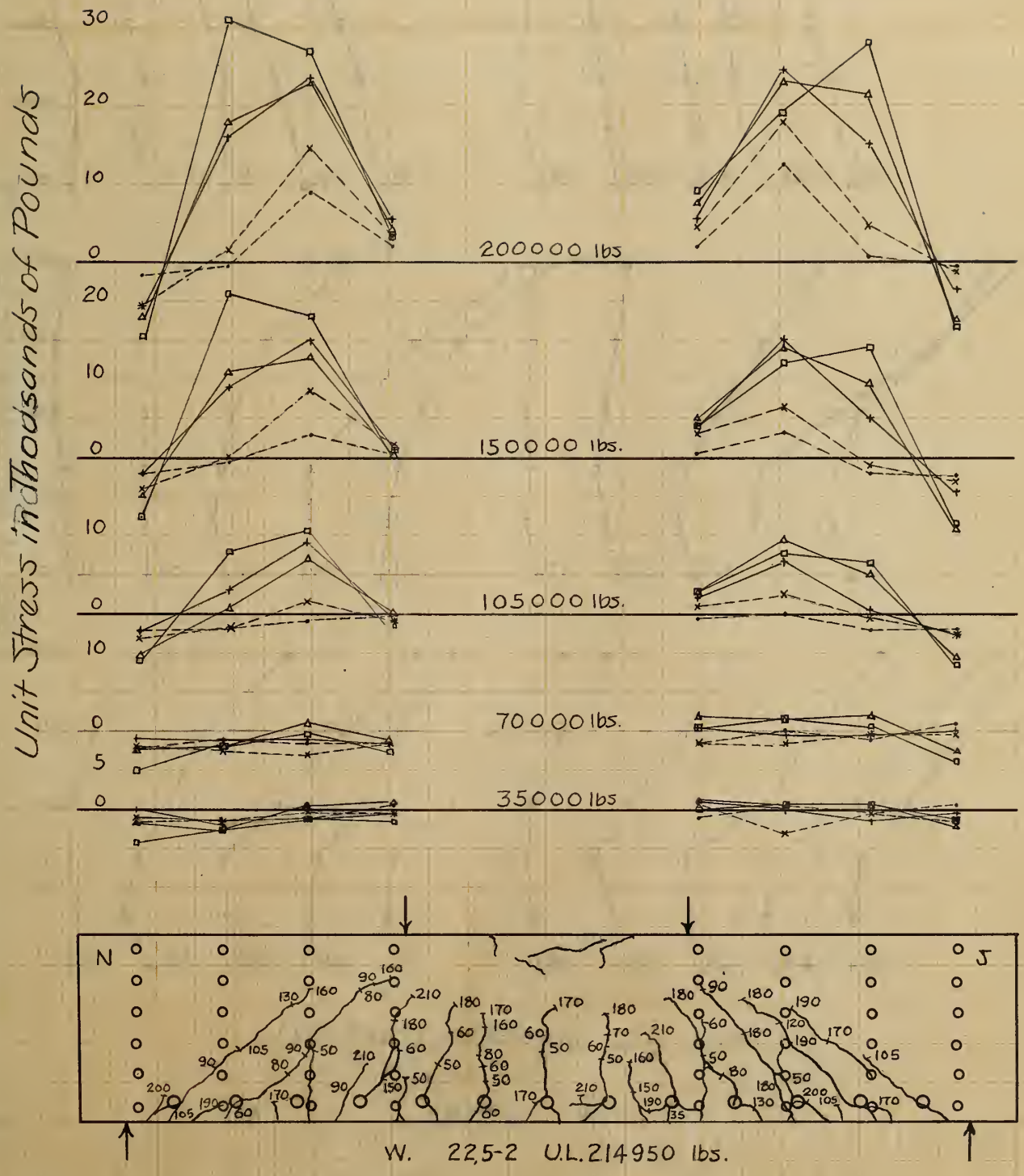
22.5-1 E.

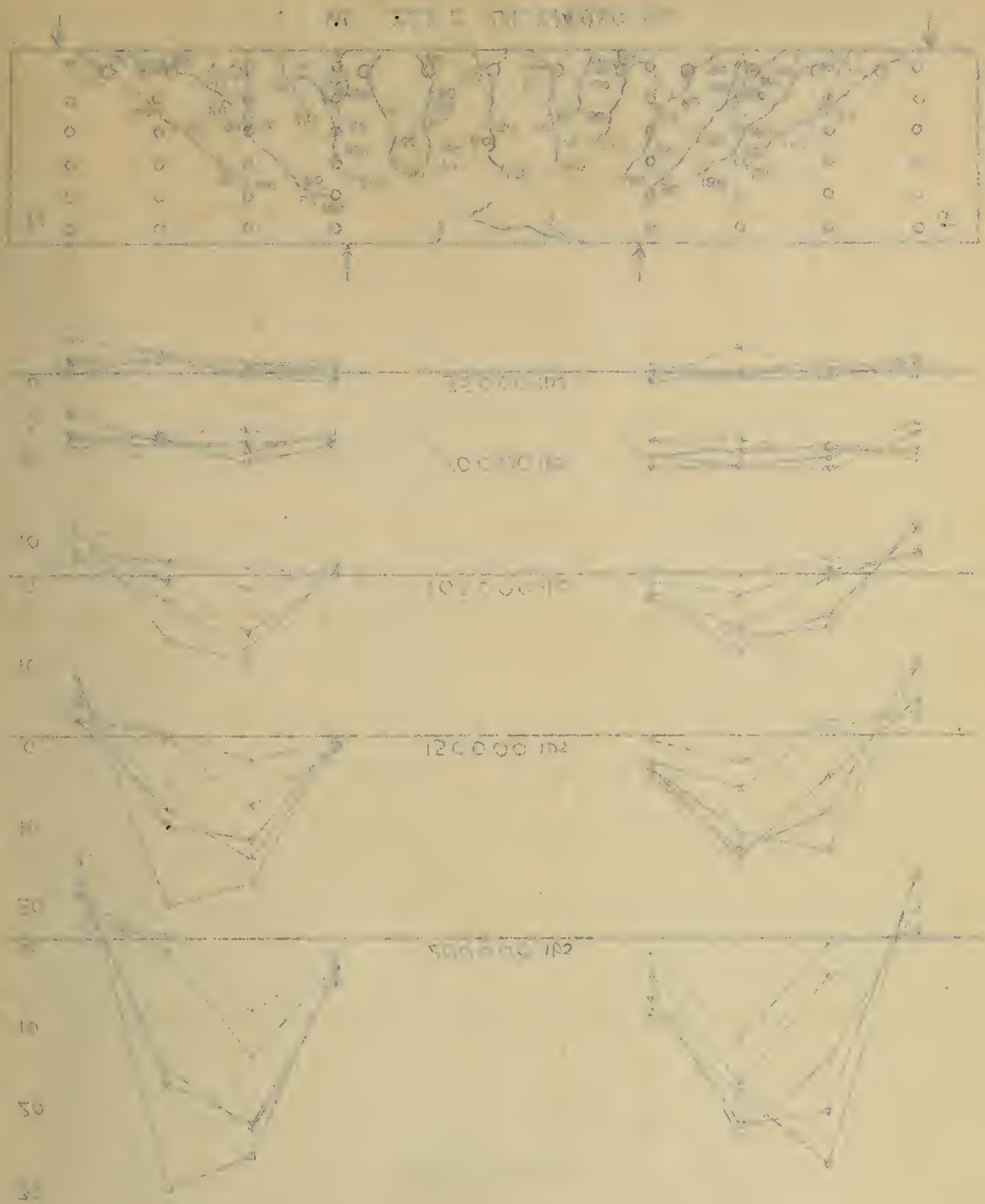


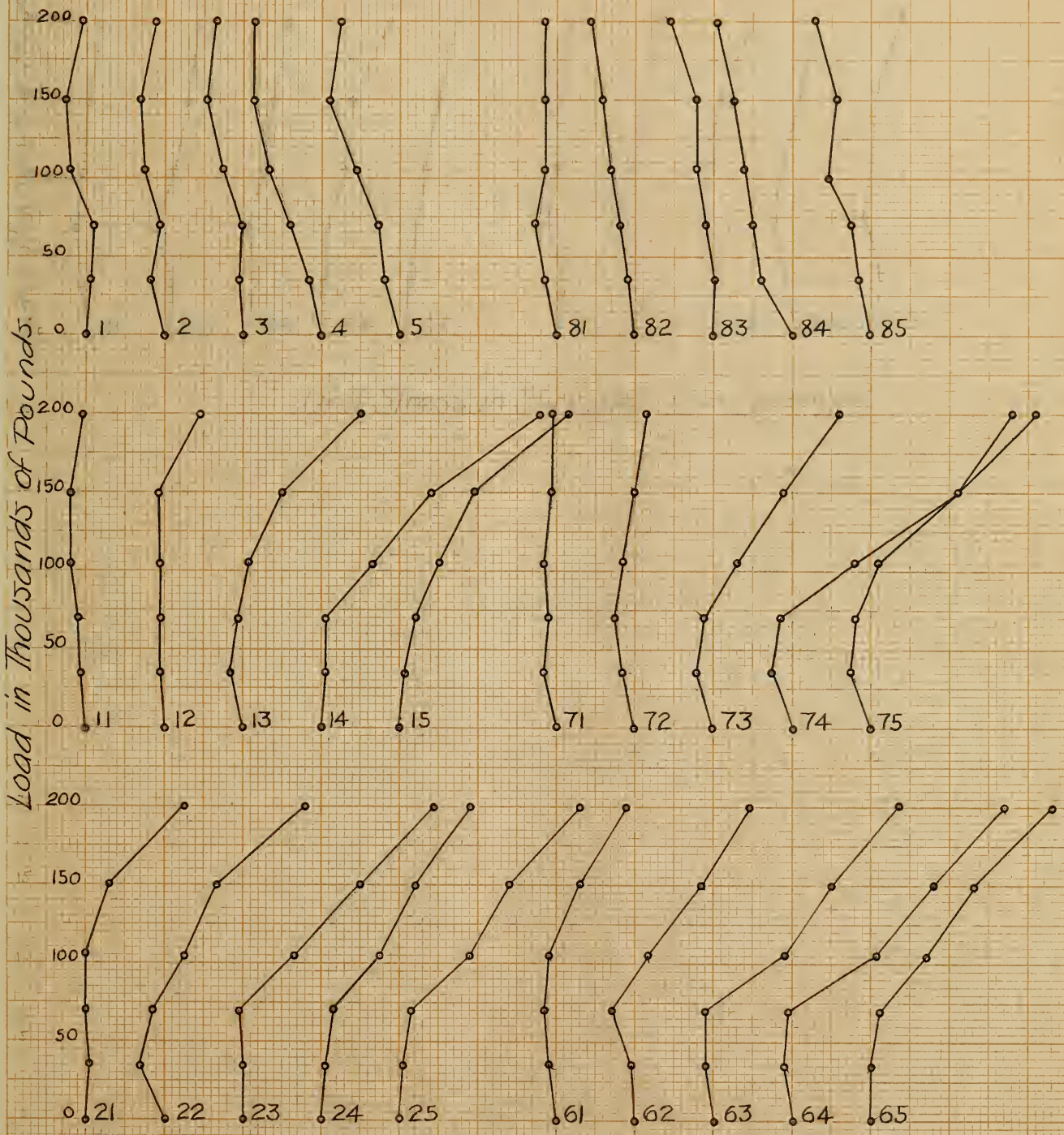
Unit Stress in Stirrups. 1 in = 20000 lbs.

22,5-1 E.


Unit Stress in Thousands of Pounds
55
50
45
40
35
30
25
20
15
10
5
0
200000 lbs.
150000 lbs.
105 000 lbs
70000 lbs
35000 lbs
N
J
W. 22,5-2 U.L. 214950 lbs.

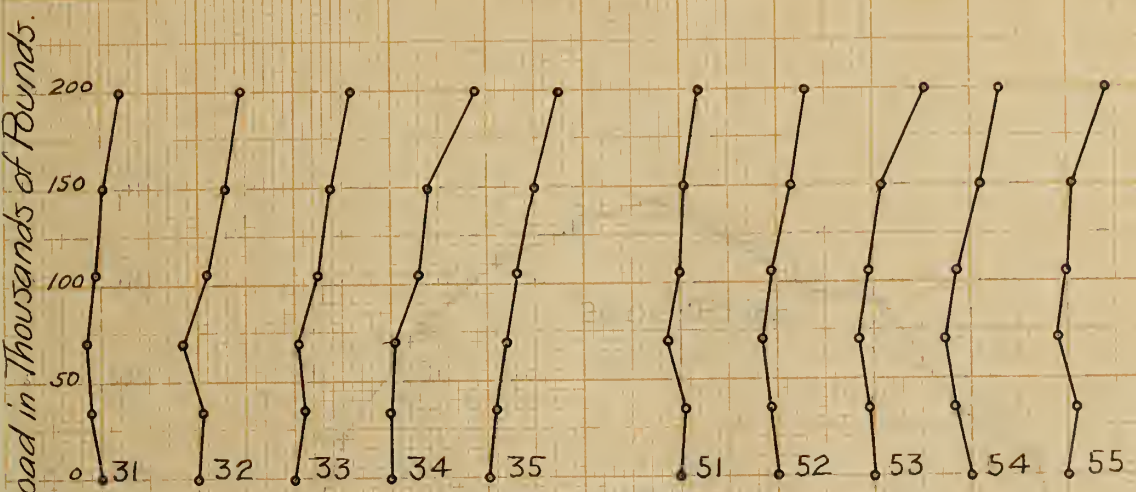

Unit Stress in Thousands of Pounds
30
20
10
0
200000 lbs
20
10
0
150000 lbs.
10
0
105000 lbs.
10
0
70000 lbs.
0
5
0
35000 lbs
N
J
W. 22,5-2 U.L. 214950 lbs.



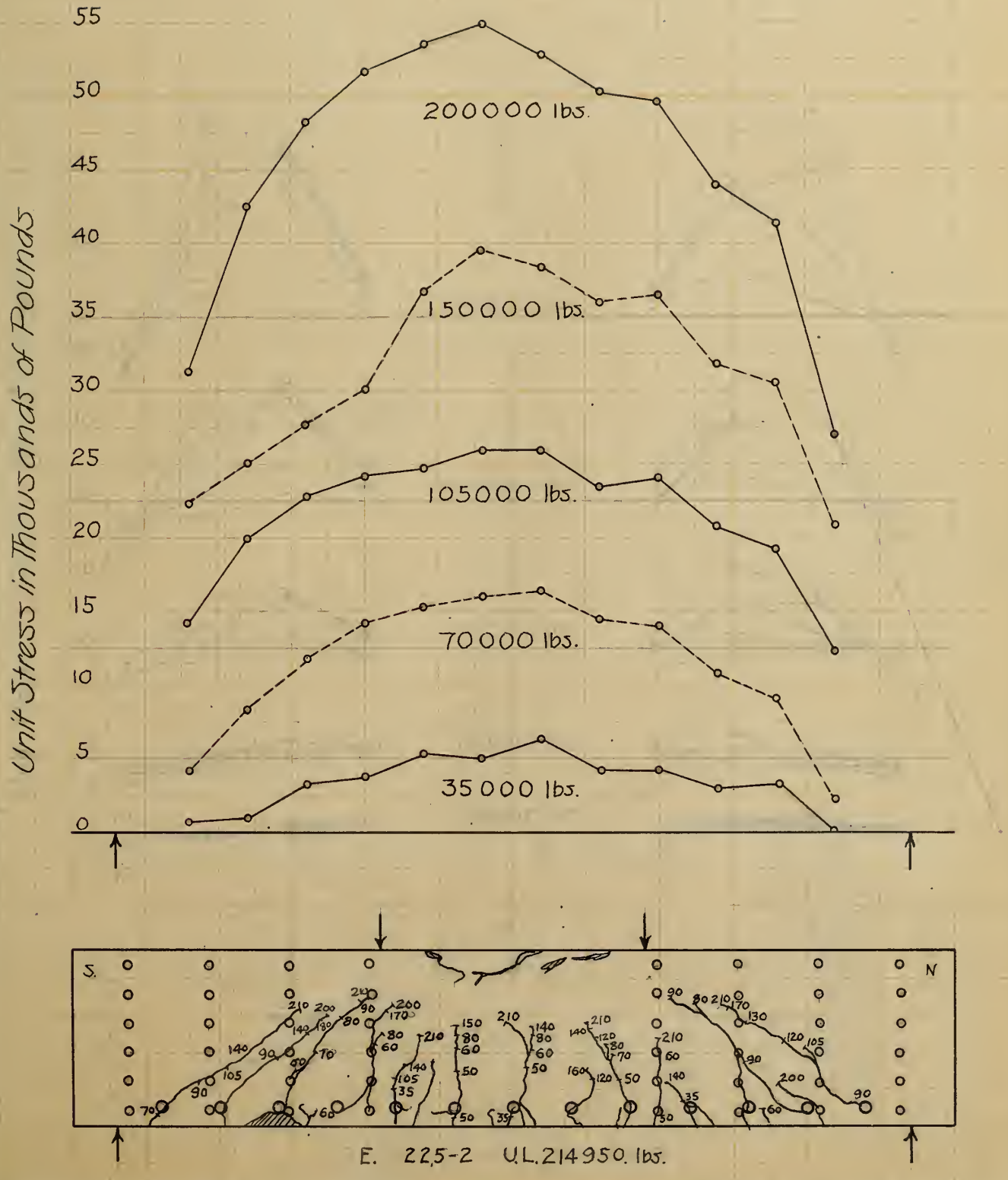


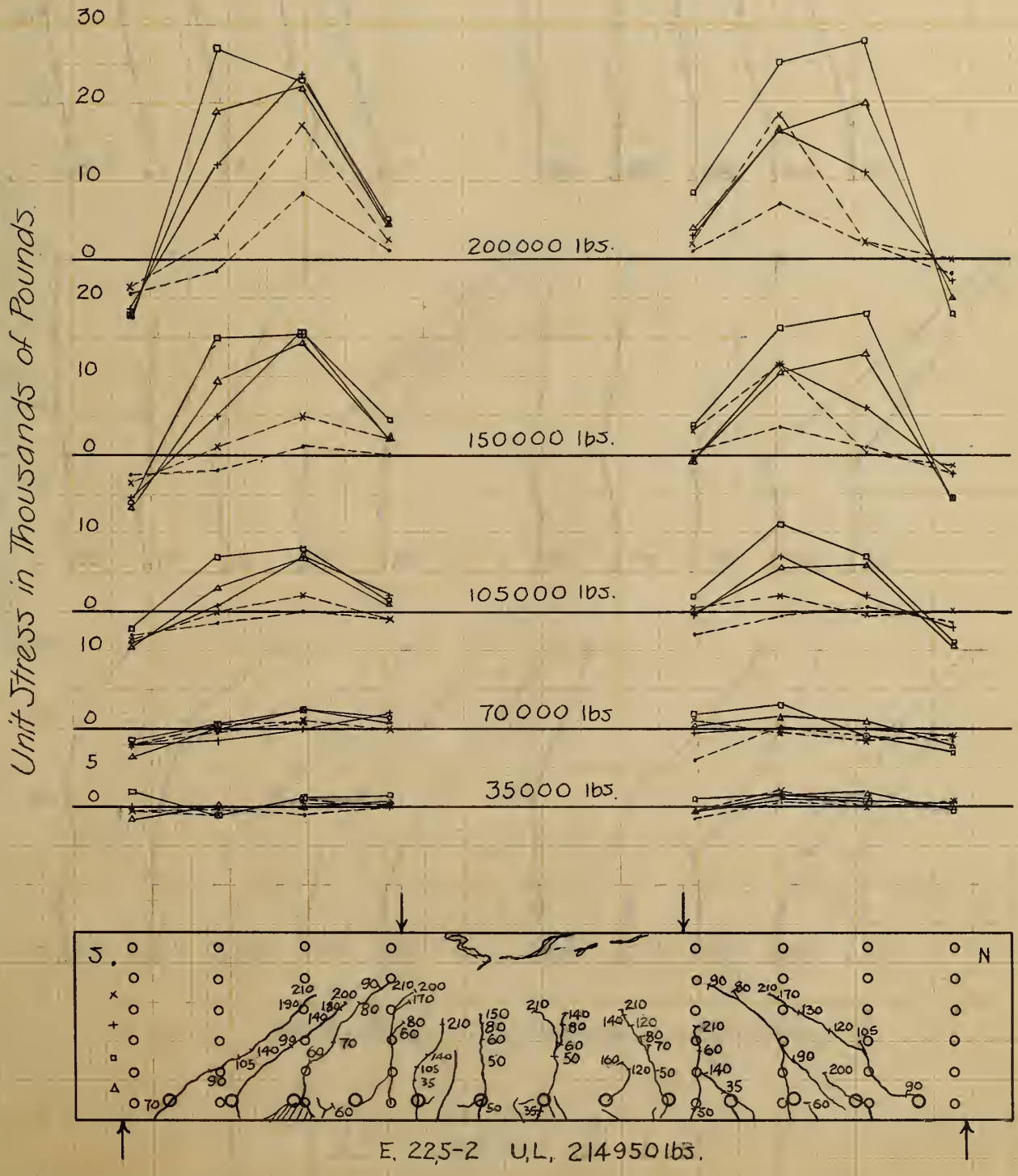
Unit Stress in Stirrups. 1 in. = 20000 lbs.

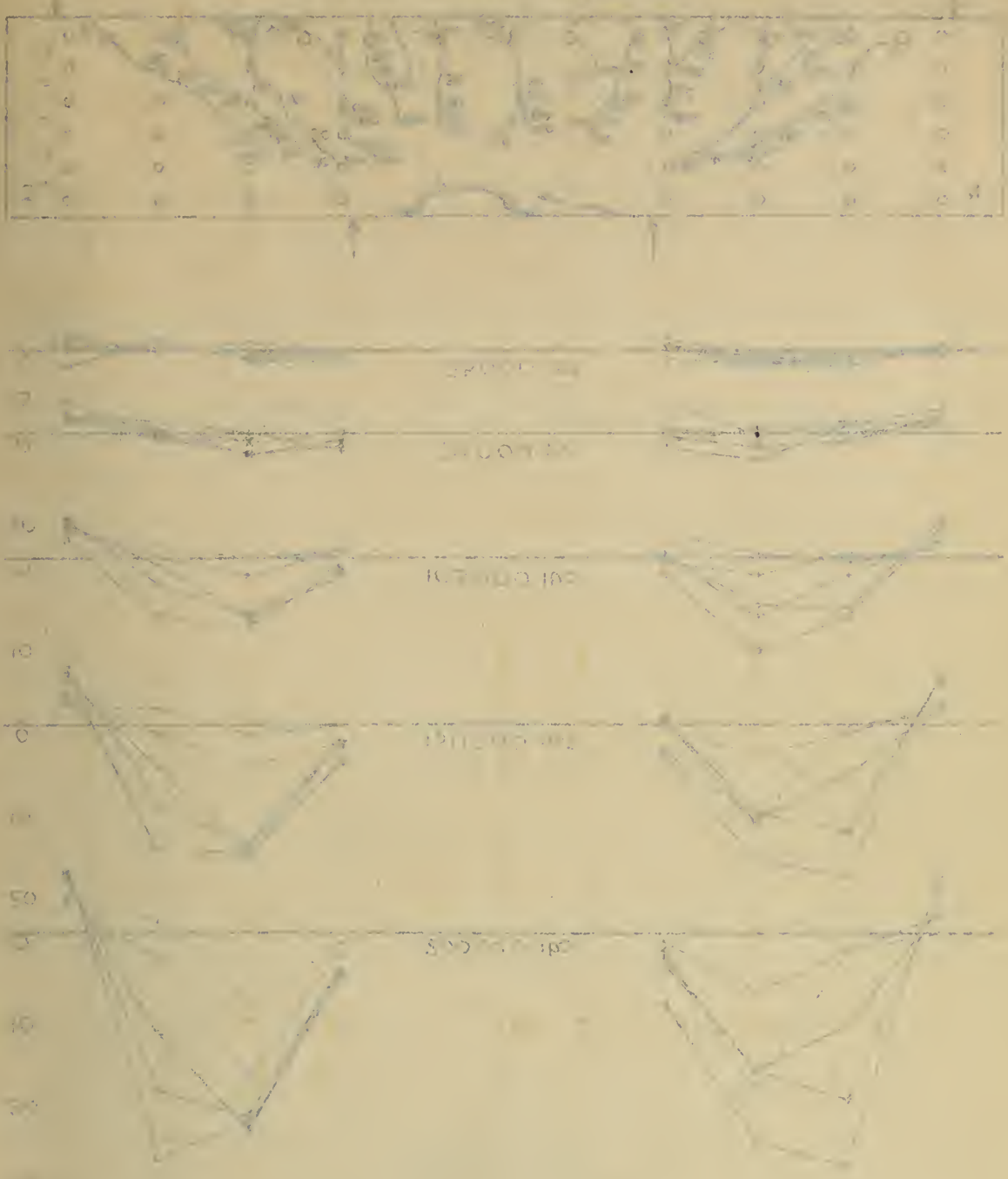
22;5-2 W

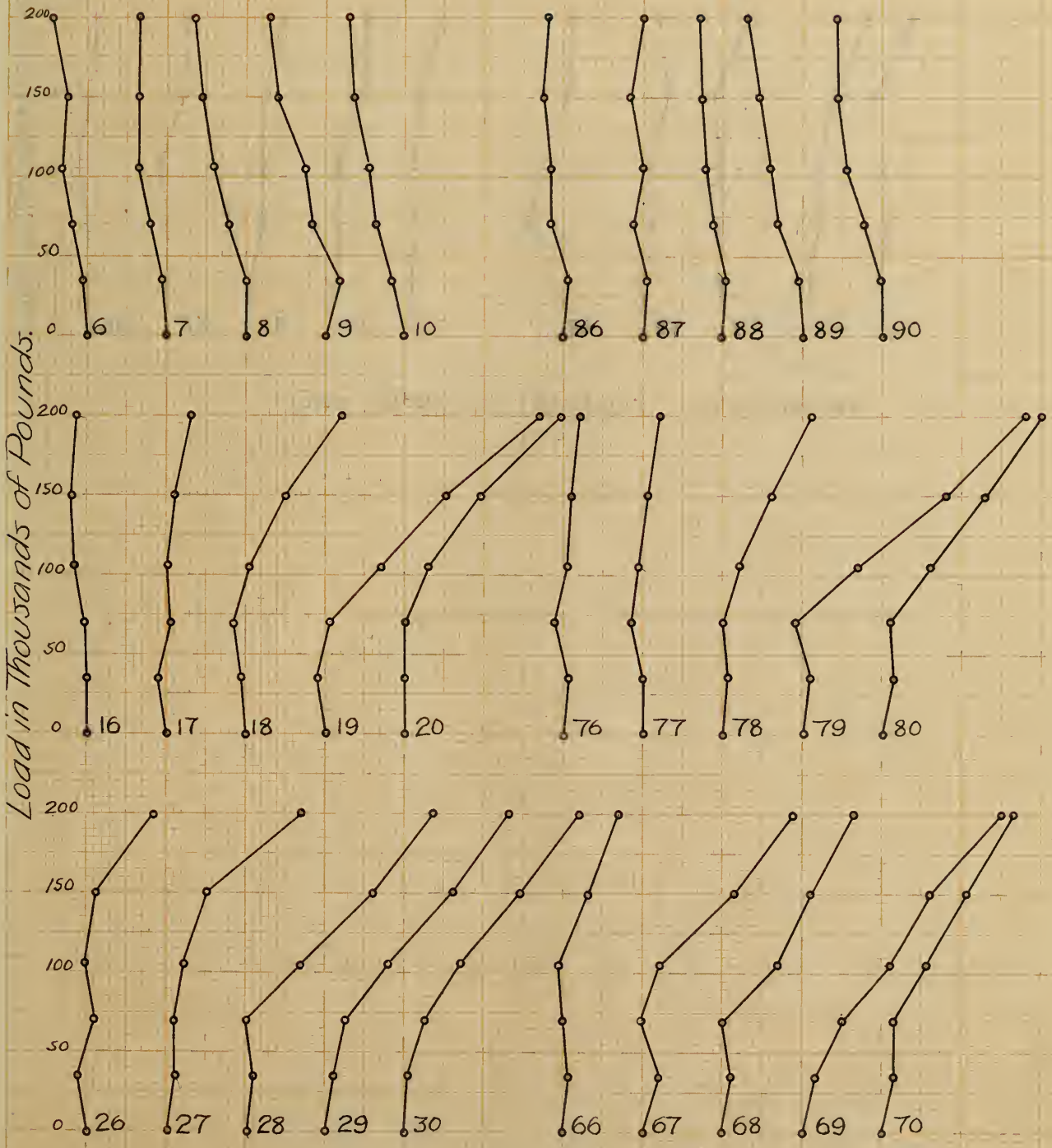


Unit Stress in Stirrups. 1 in. = 20000 lbs.


Unit Stress in Thousands of Pounds
55
50
45
40
35
30
25
20
15
10
5
0
200000 lbs.
150000 lbs.
105000 lbs.
70000 lbs.
35000 lbs.
S.
N
E. 22,5-2 U.L. 214950. lbs.

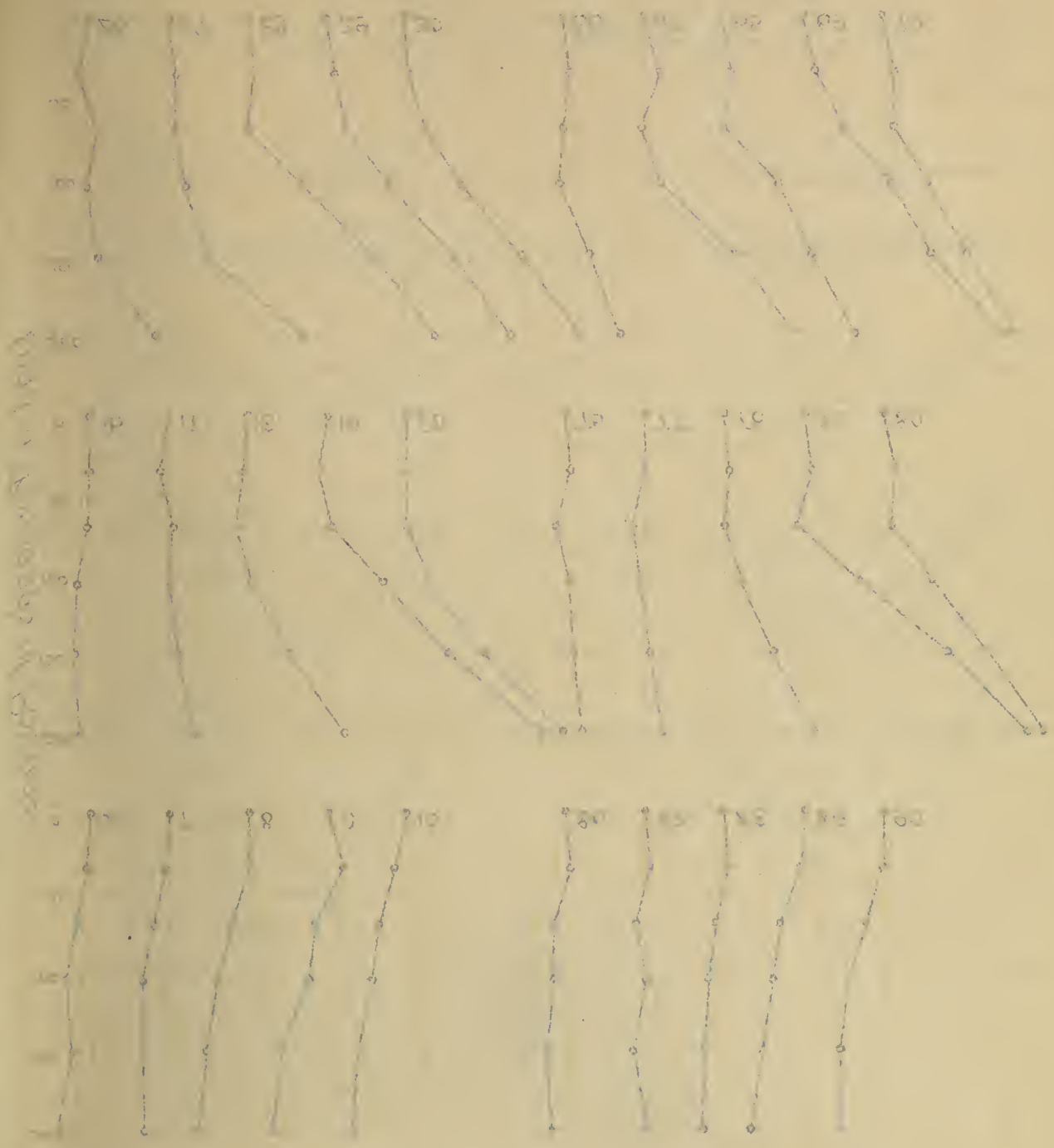

Unit Stress in Thousands of Pounds
30
20
10
0
200000 lbs.
20
10
0
150000 lbs.
10
0
105000 lbs.
10
0
70000 lbs
5
0
35000 lbs.
S.
N
E. 22,5-2 U.L. 214950 lbs.

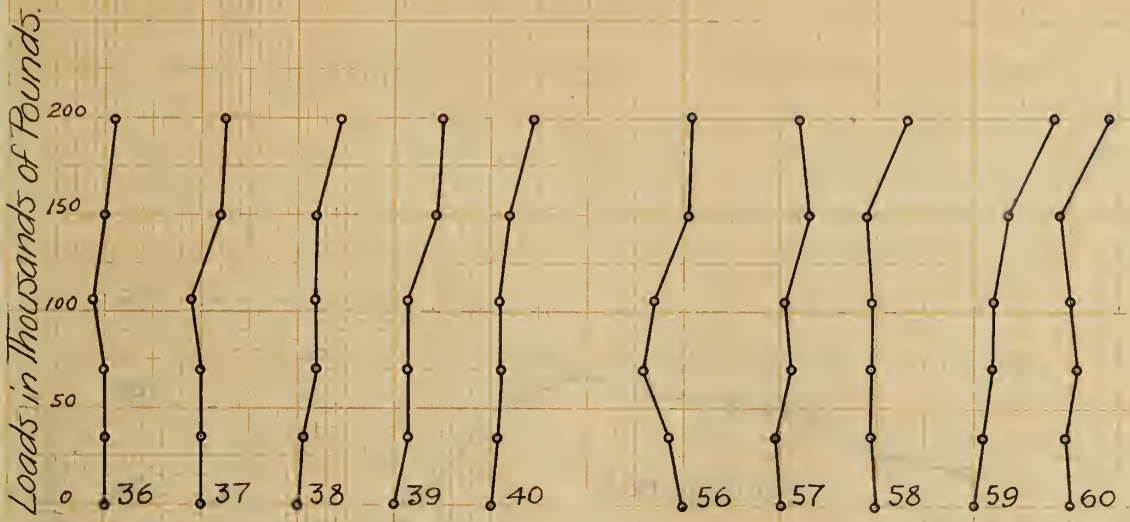




Unit Stress in Stirrups. 1 in. = 20000 lbs.

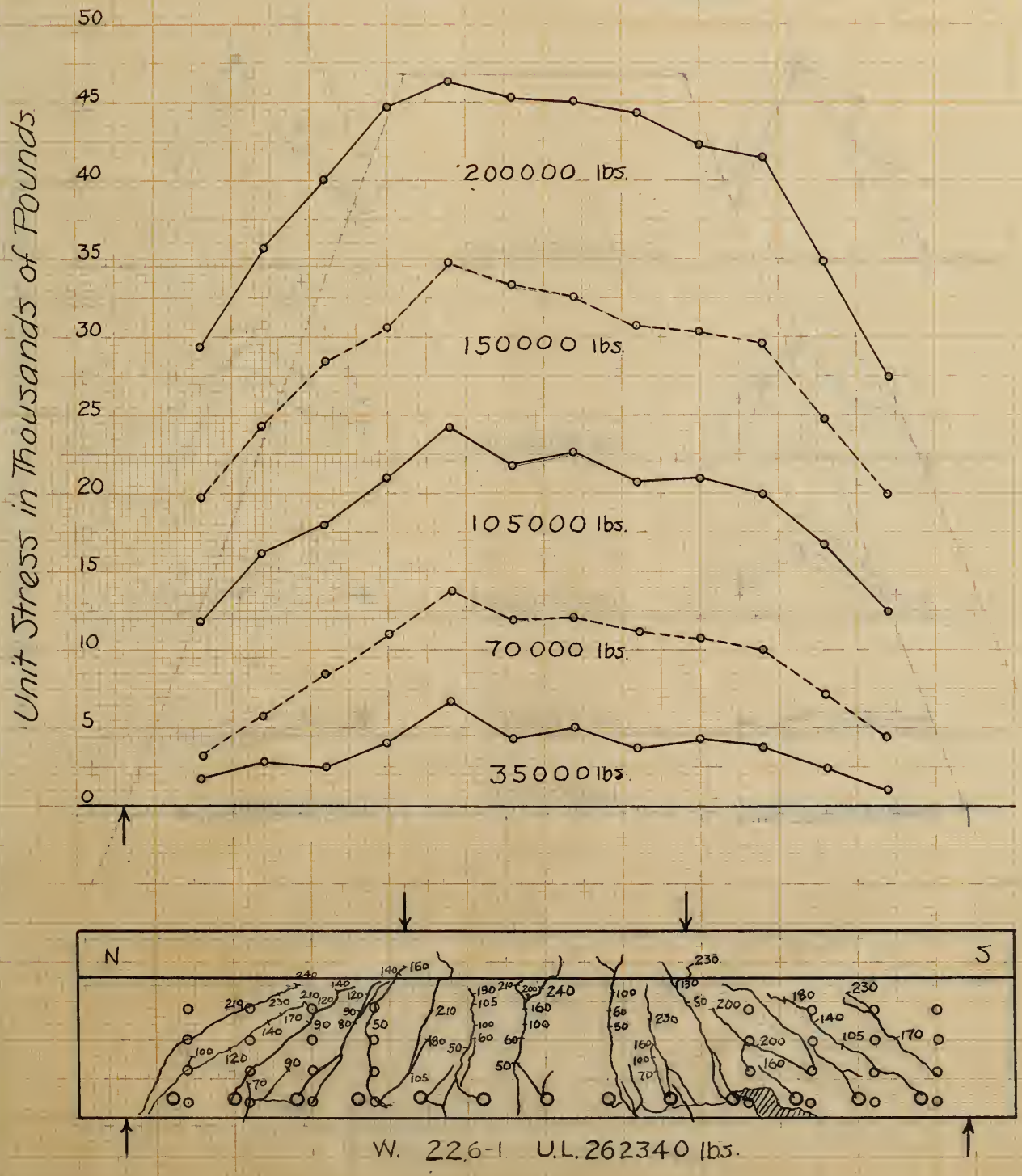
22,5-2. E.



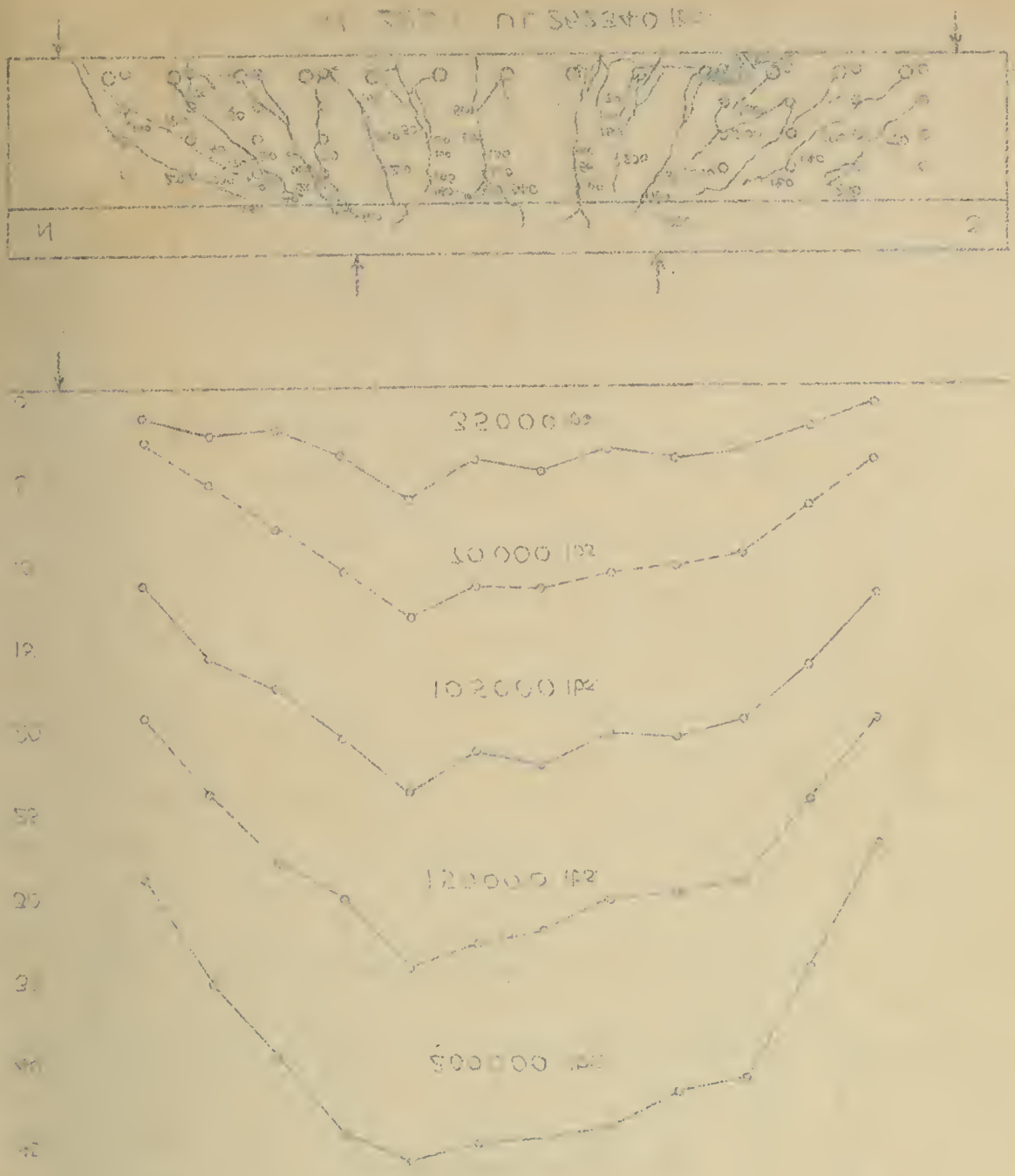


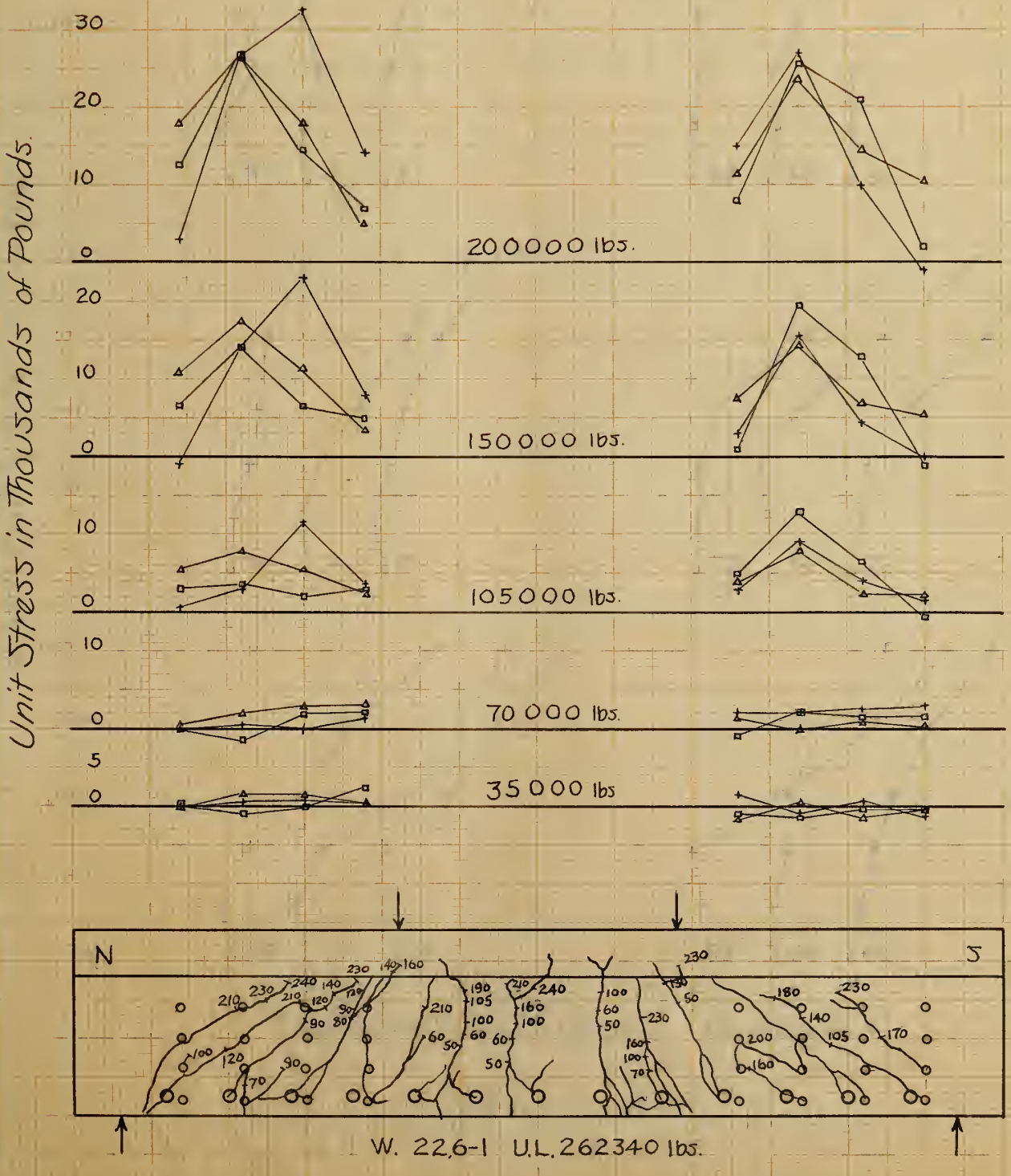
Unit Stress in Stirrups 1 in. = 20000 lbs.

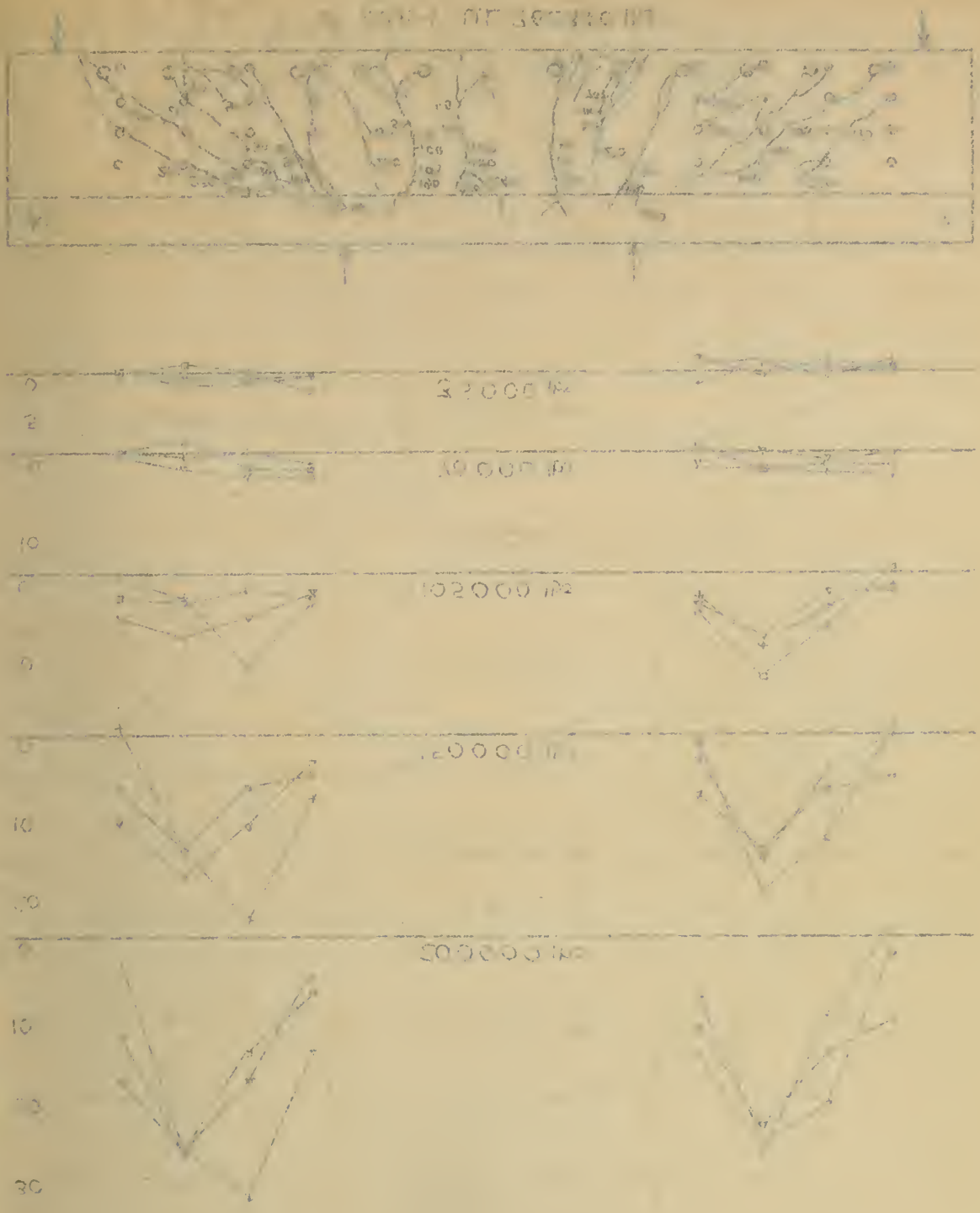
22.5-2. E,

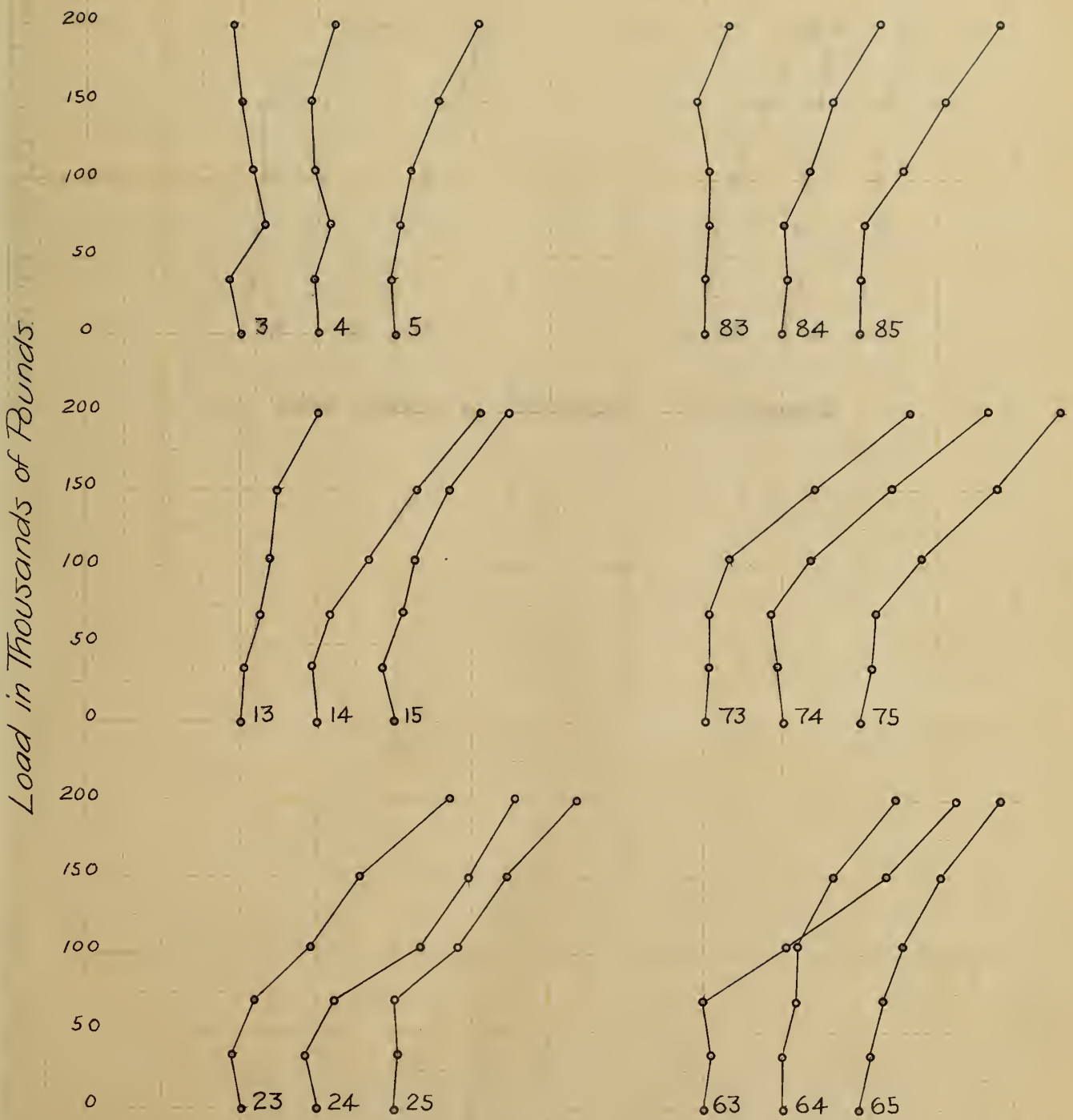


W. 22.6-1 U.L. 262340 lbs.

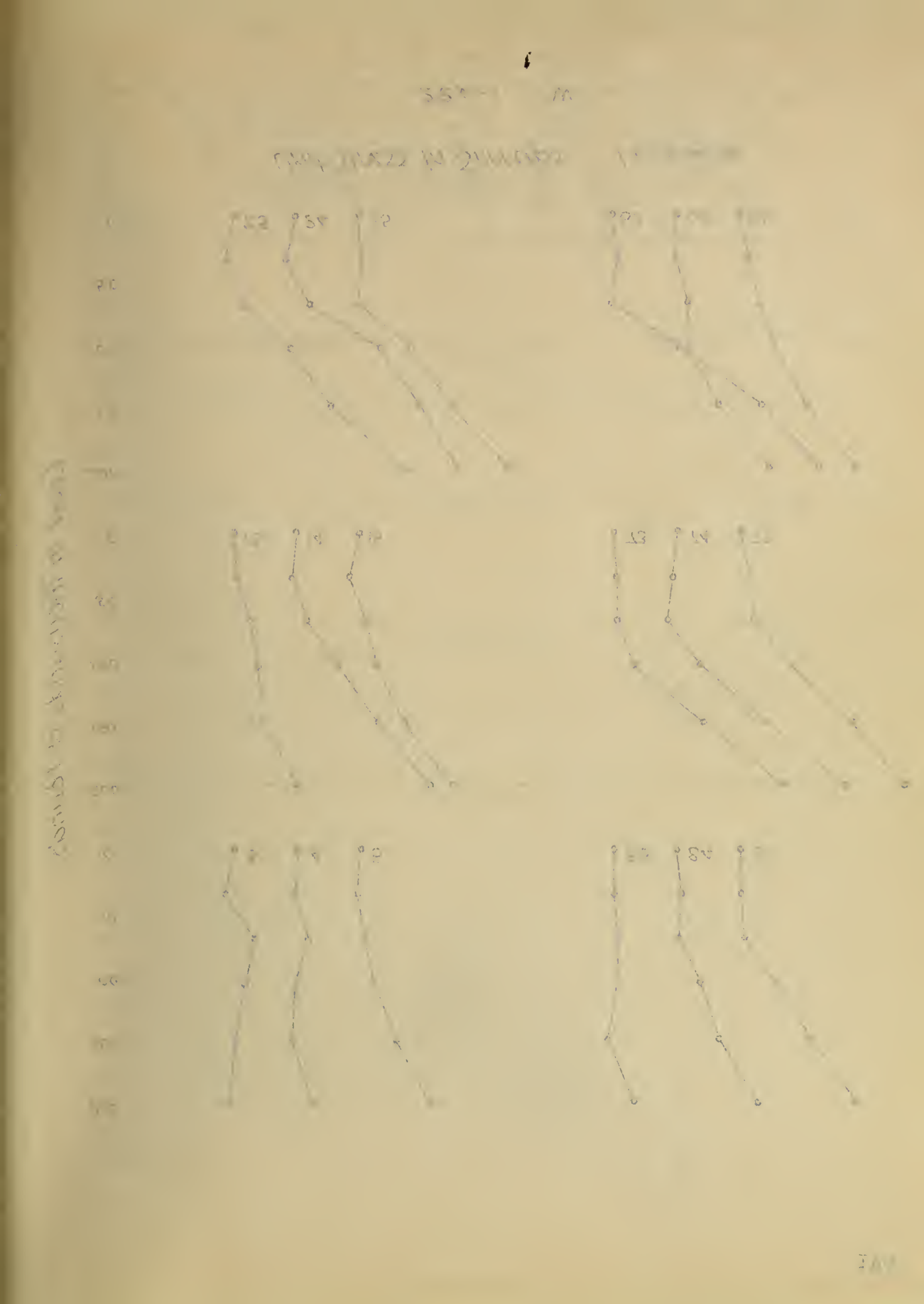


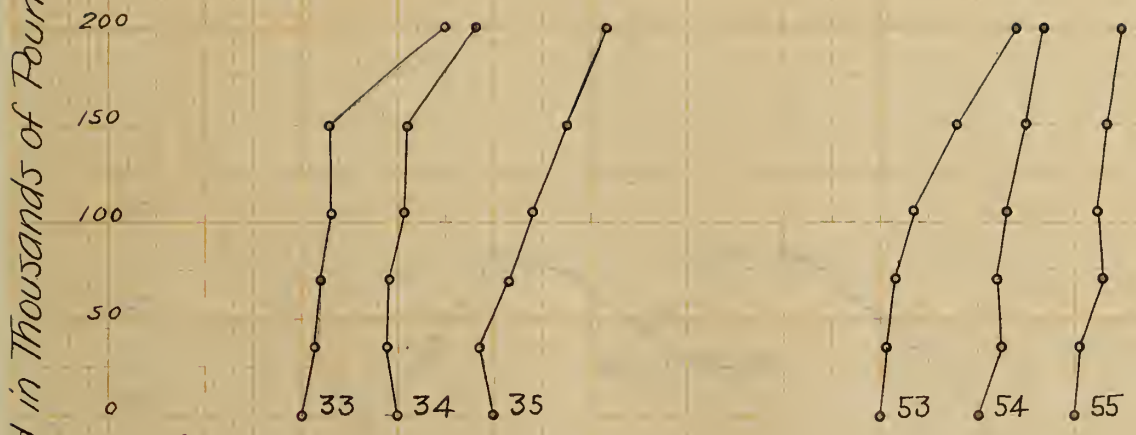

Unit Stress in Thousands of Pounds.
30
20
10
0
200000 lbs.
20
10
0
150000 lbs.
10
0
105000 lbs.
10
0
70000 lbs.
5
0
35000 lbs
N
S
W. 22.6-1 U.L. 262340 lbs.



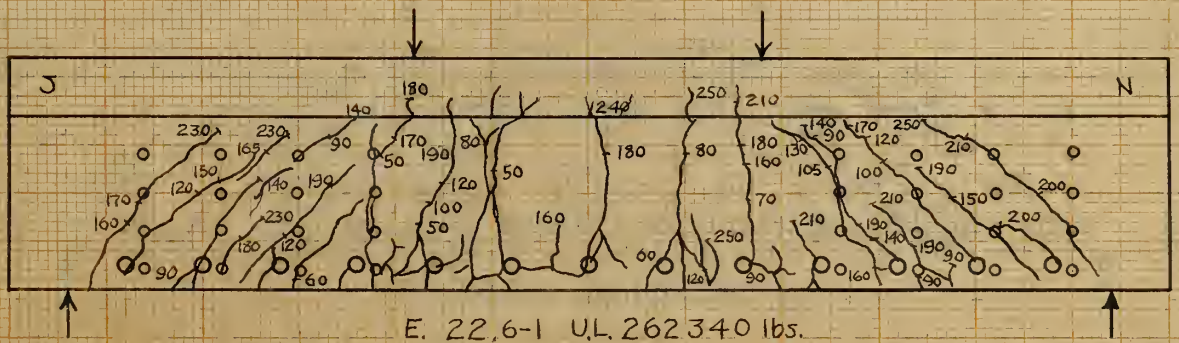

Load in Thousands of Pounds.
200
150
100
50
0
3
4
5
83
84
85
13
14
15
73
74
75
23
24
25
63
64
65
Unit Stress in Stirrups.
1 in = 20000 lbs.

22,6-1 W.

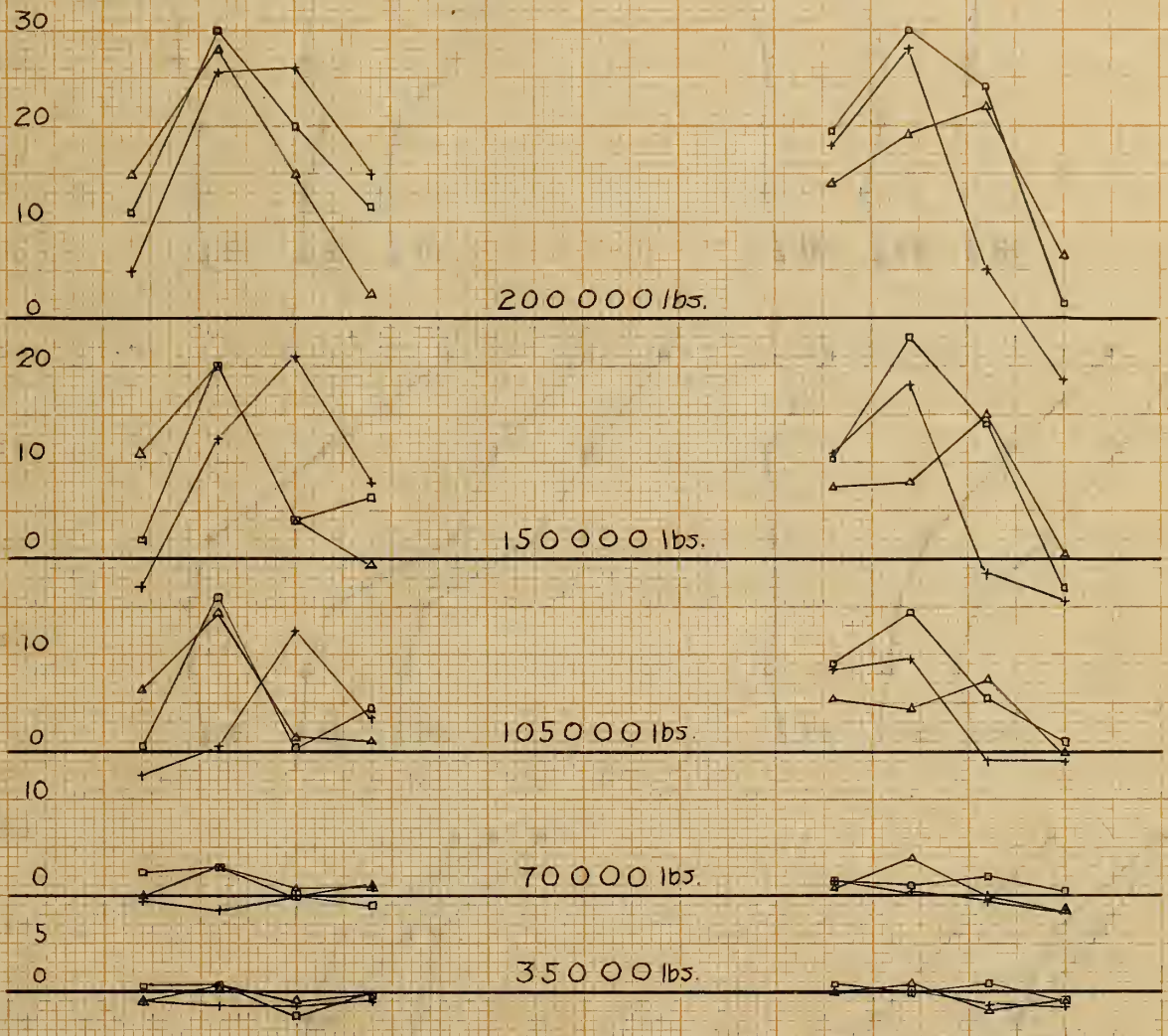


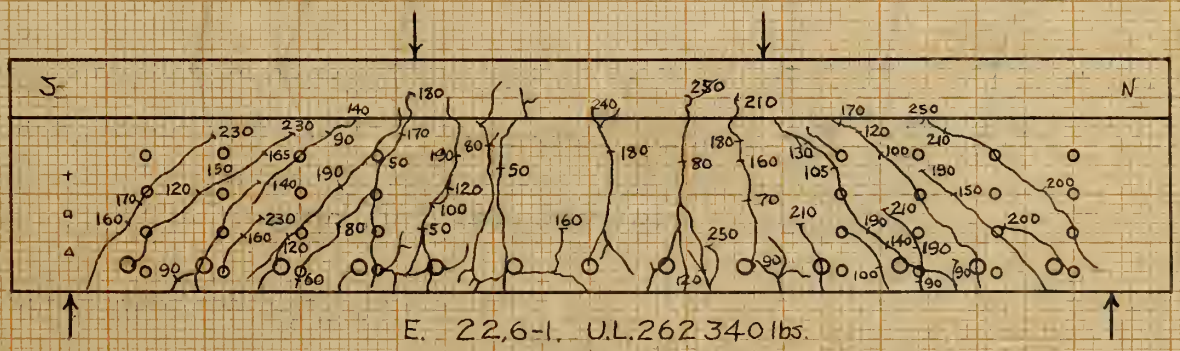


Unit Stress in Stirrups. 1 in. = 20000 lbs.

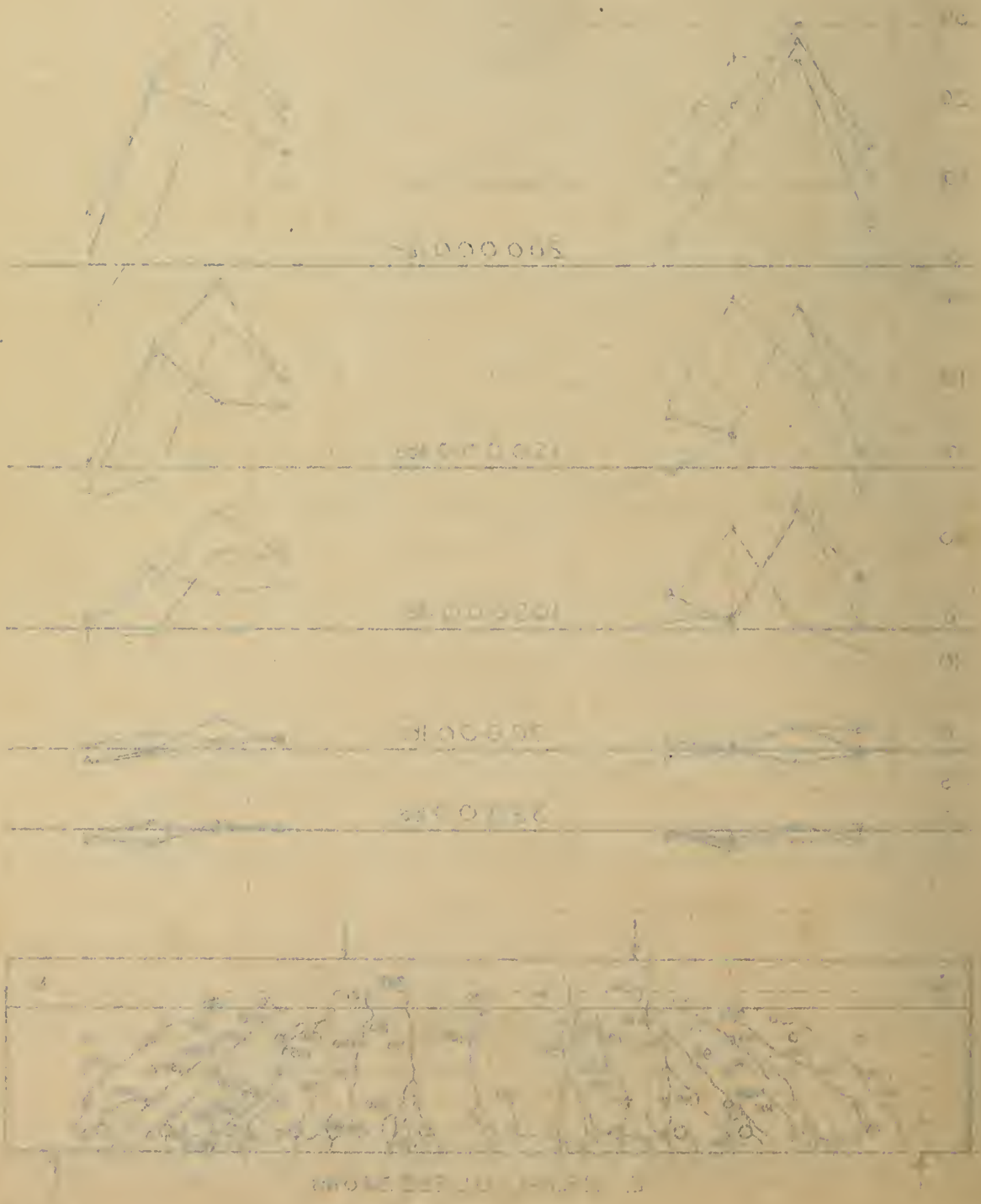


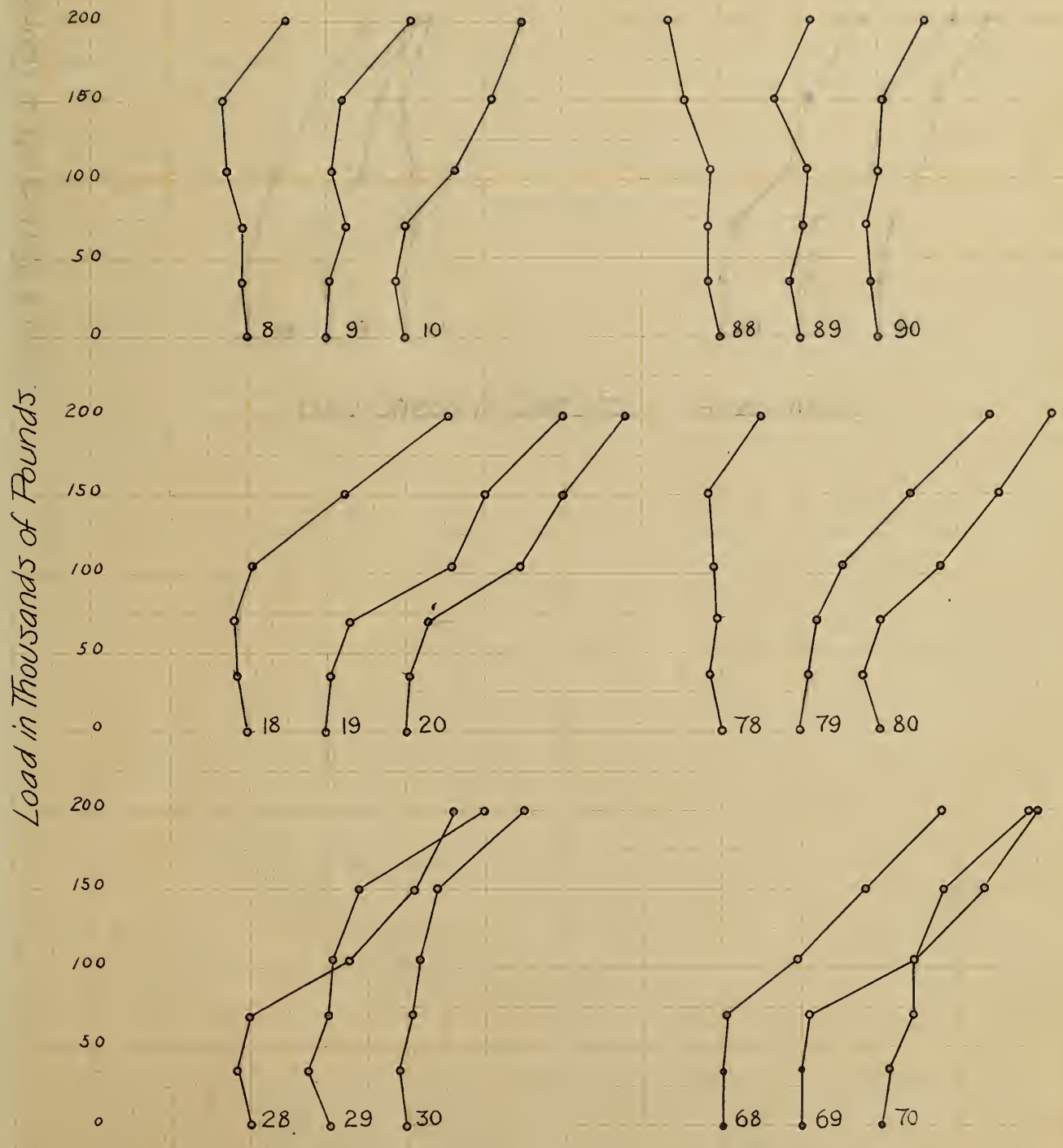
E. 22,6-1 U.L. 262340 lbs.



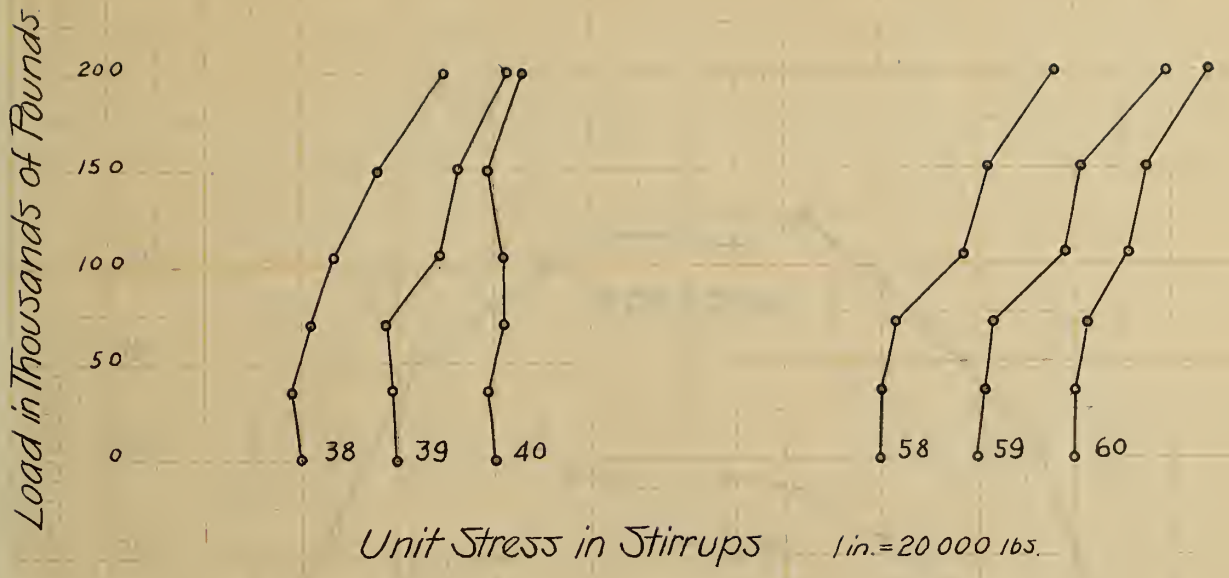


E. 22,6-1. U.L. 262340 lbs.

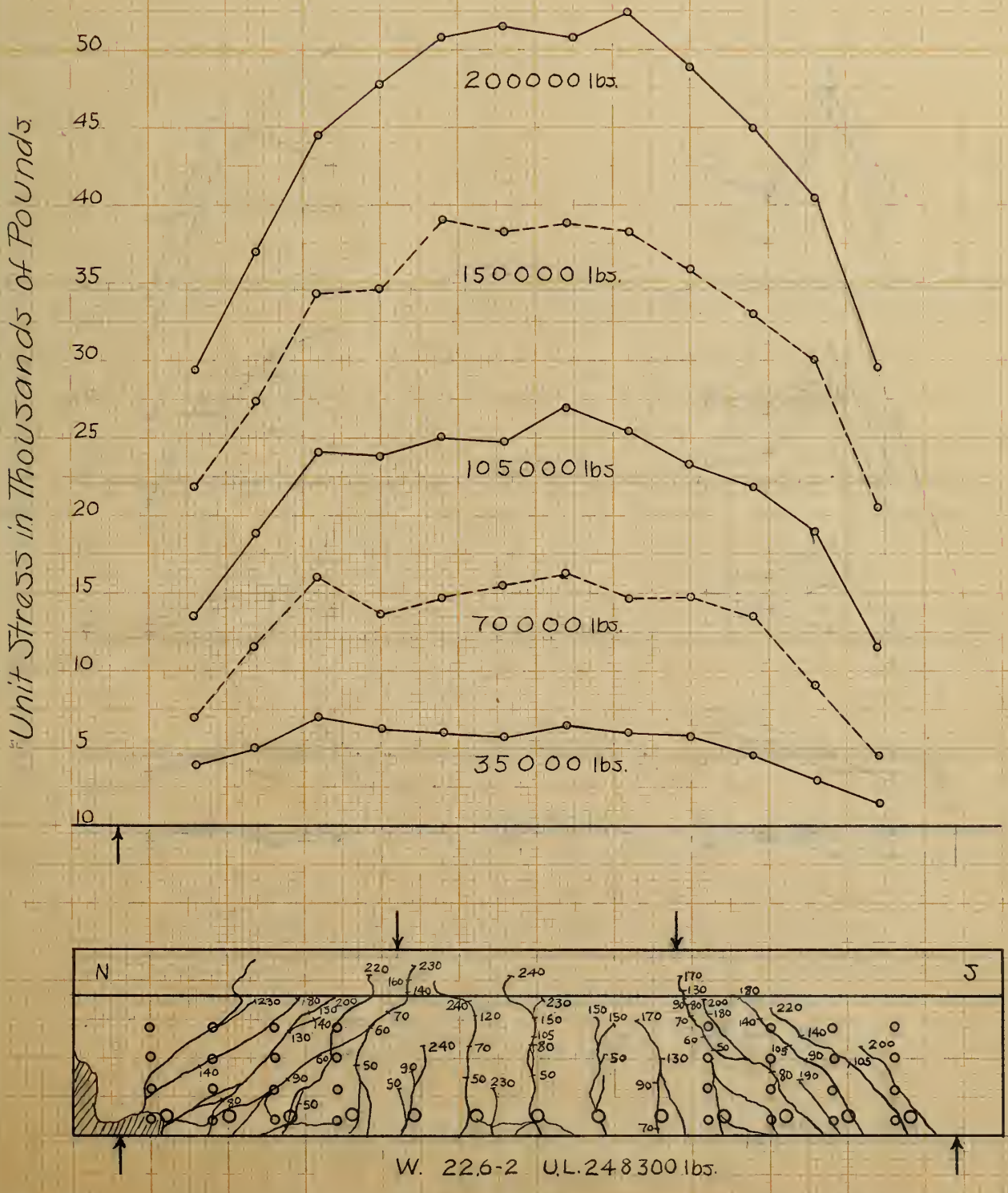


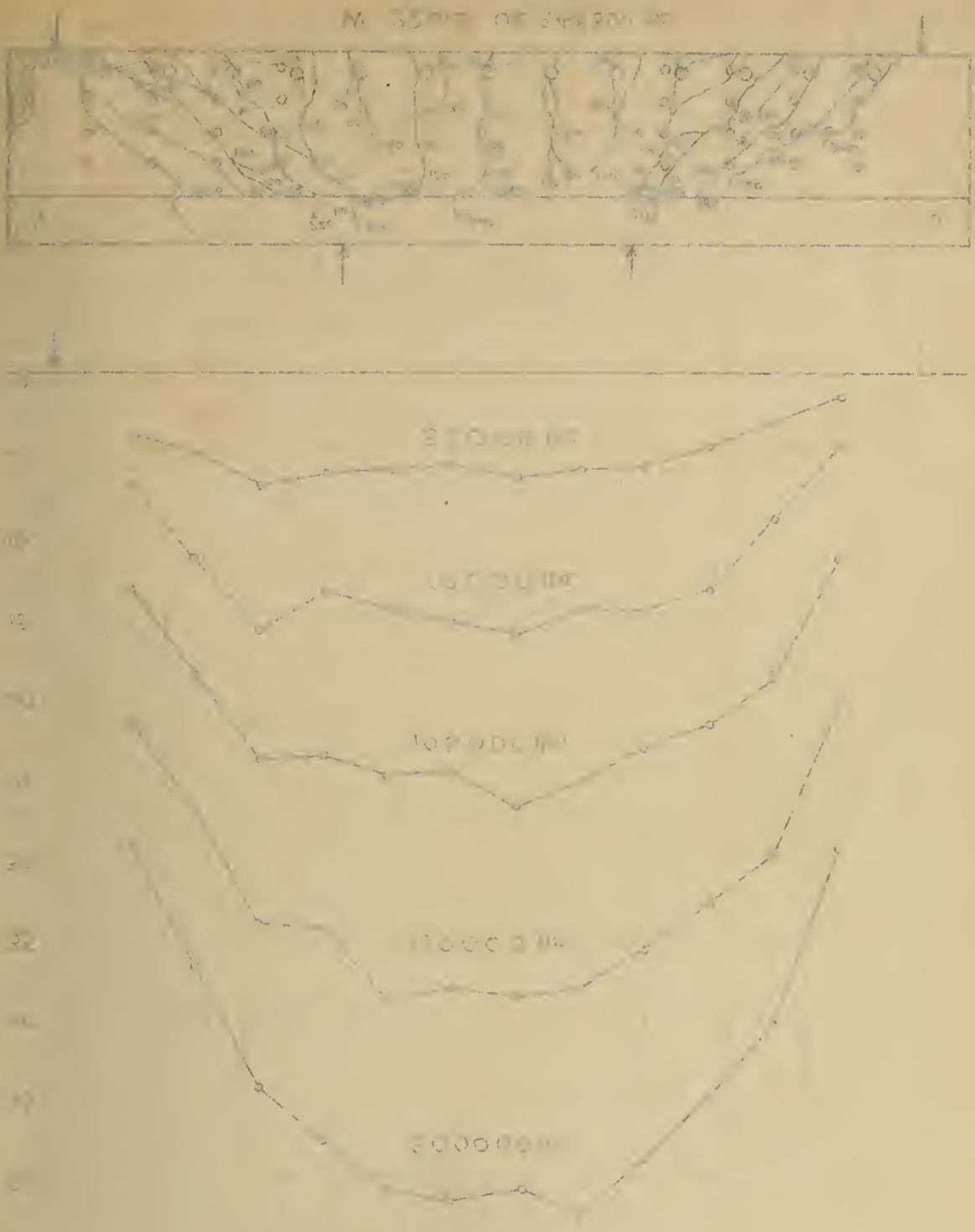

200
150
100
50
0
8
9
10
88
89
90
18
19
20
78
79
80
28
29
30
68
69
70
Load in Thousands of Pounds.
Unit Stress in Stirrups
1 in. = 20 000 lbs.

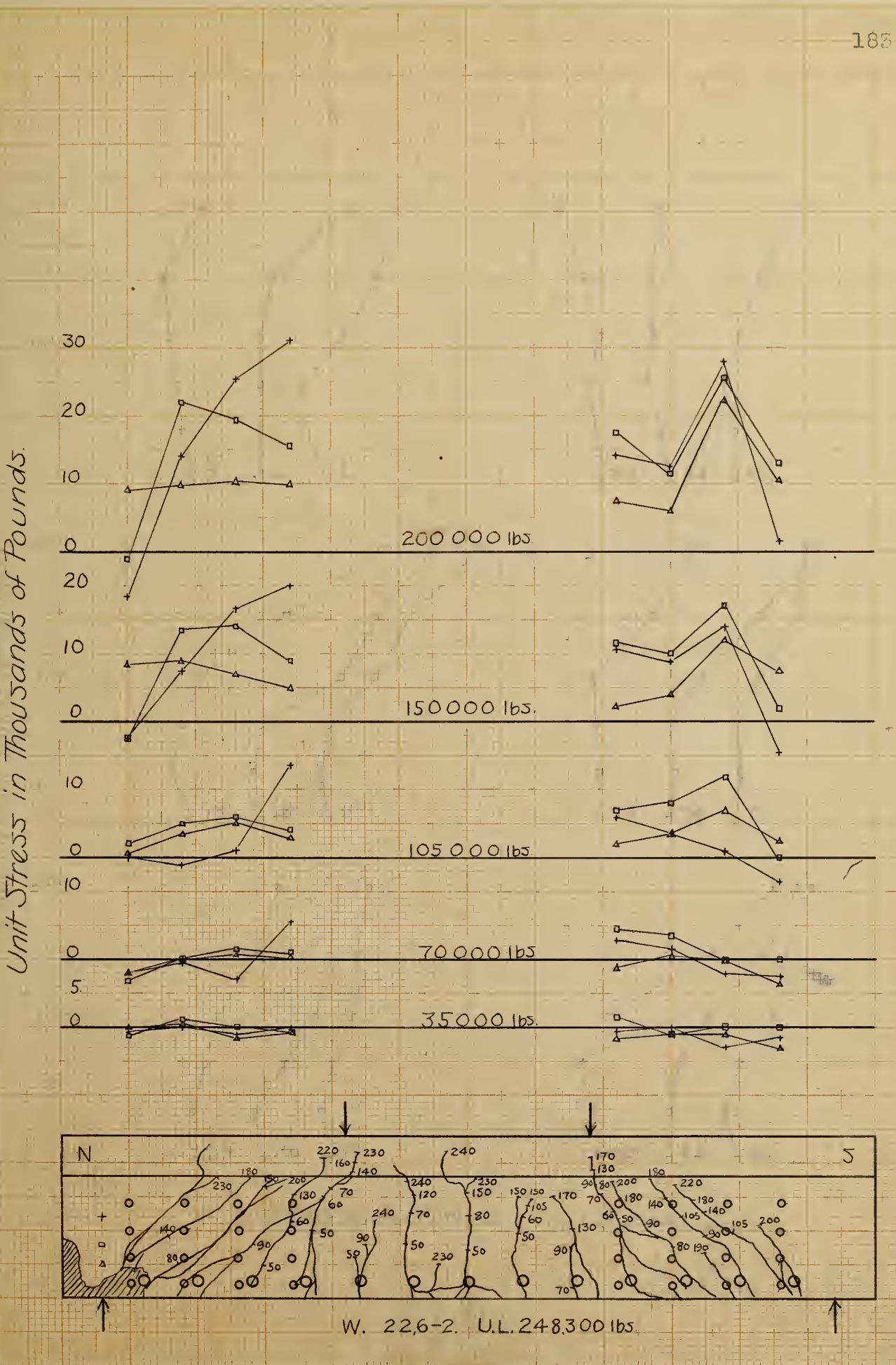
22,6-1 E.

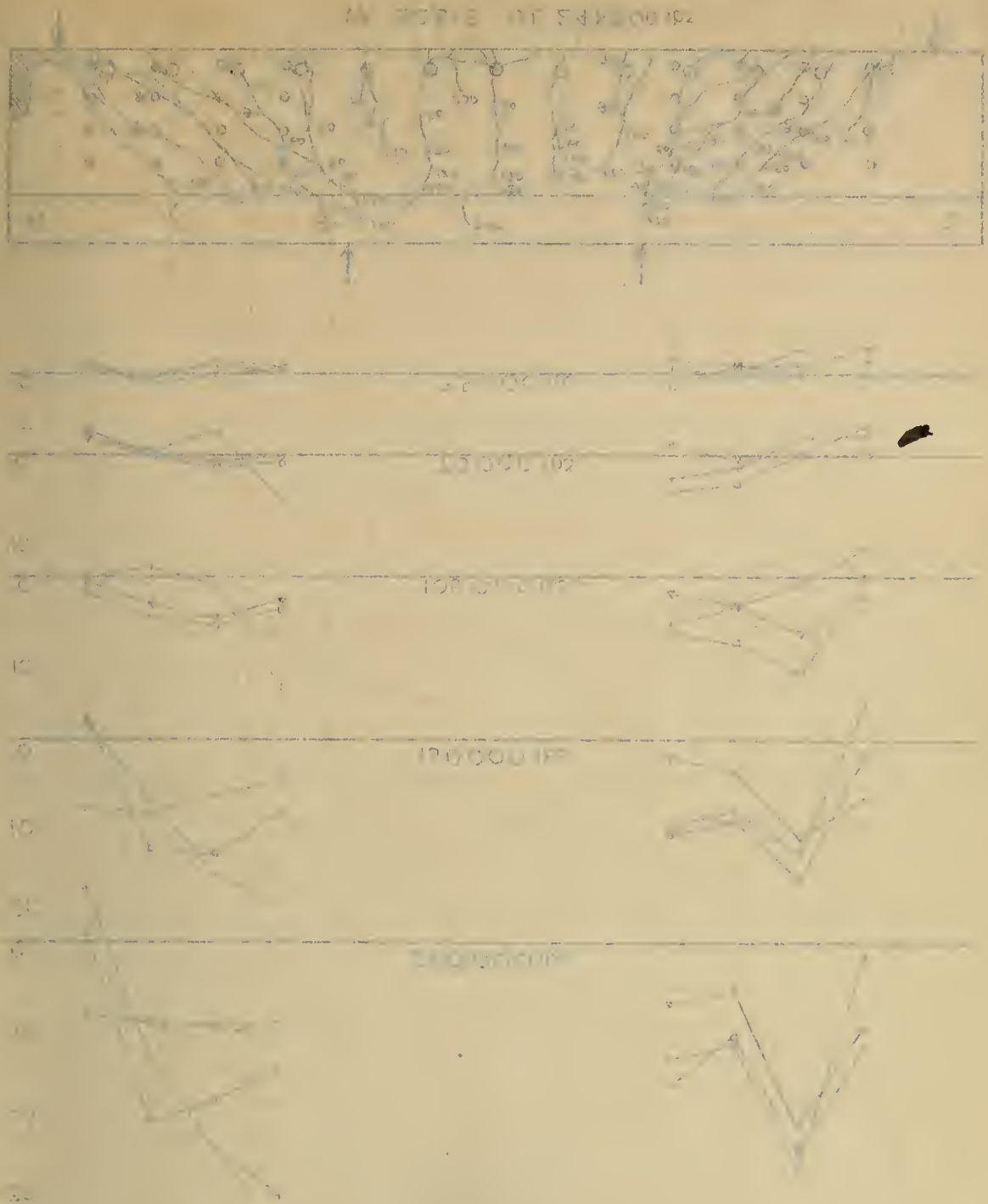


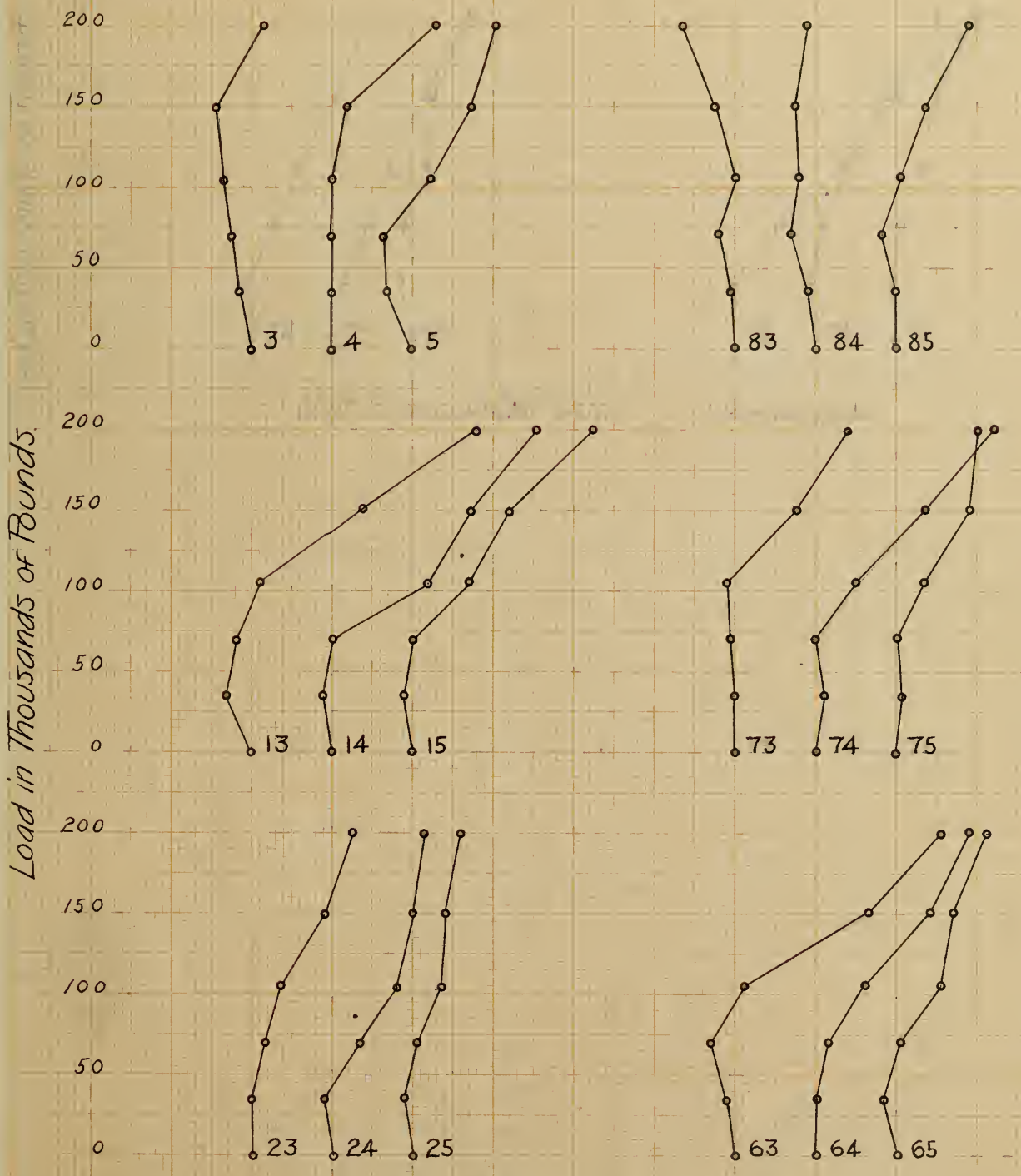
Unit Stress in Stirrups 1 in. = 20 000 lbs.


Unit Stress in Thousands of Pounds
50
45
40
35
30
25
20
15
10
5
10
200000 lbs.
150000 lbs.
105000 lbs
70000 lbs.
35000 lbs.
N
S
W. 22.6-2 U.L. 248300 lbs.



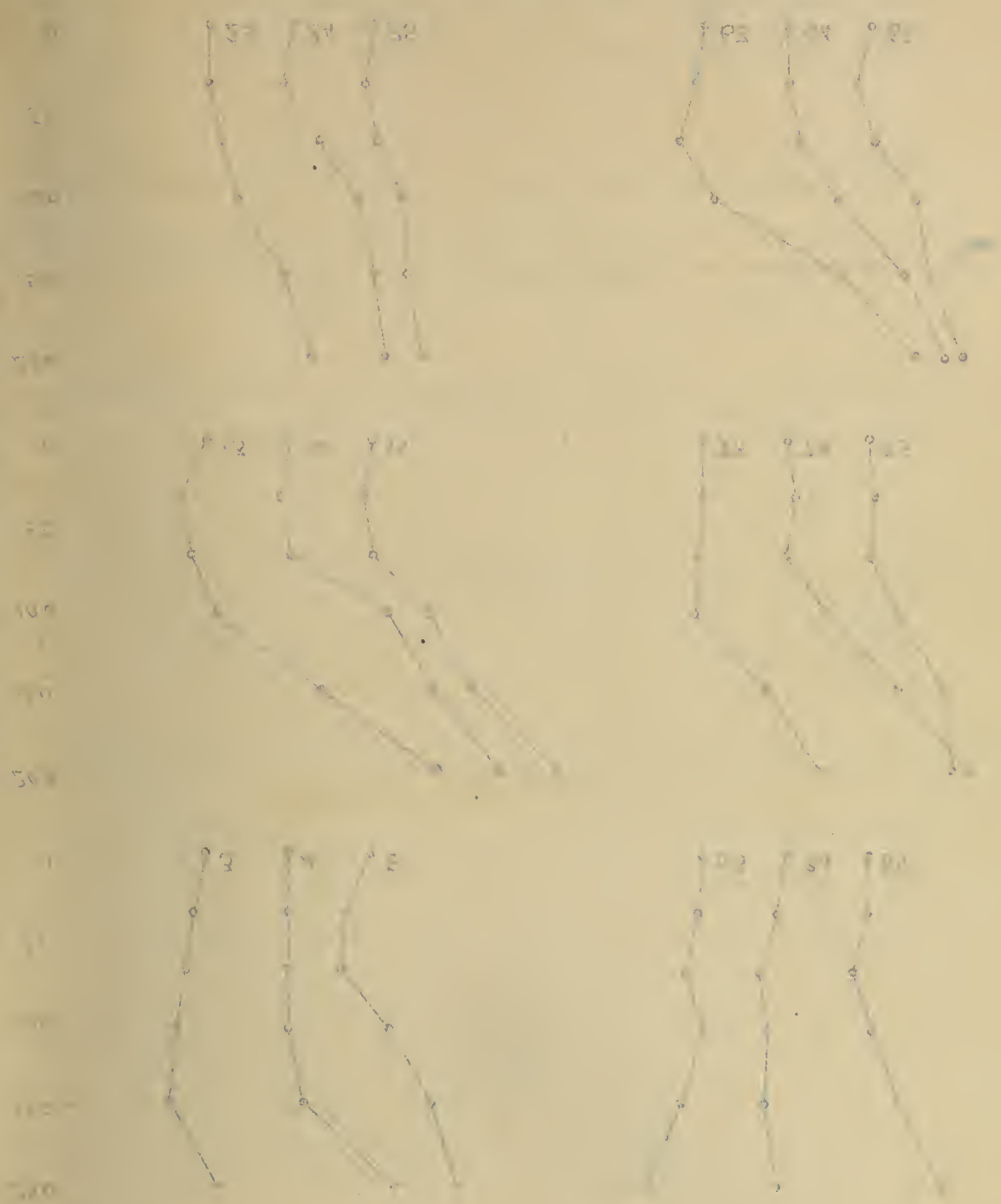

Unit Stress in Thousands of Pounds.
30
20
10
0
200 000 lbs.
20
10
0
150000 lbs.
10
0
105 000 lbs.
10
0
70000 lbs
5.
0
35000 lbs.
N
S
W. 22,6-2. U.L. 248,300 lbs.

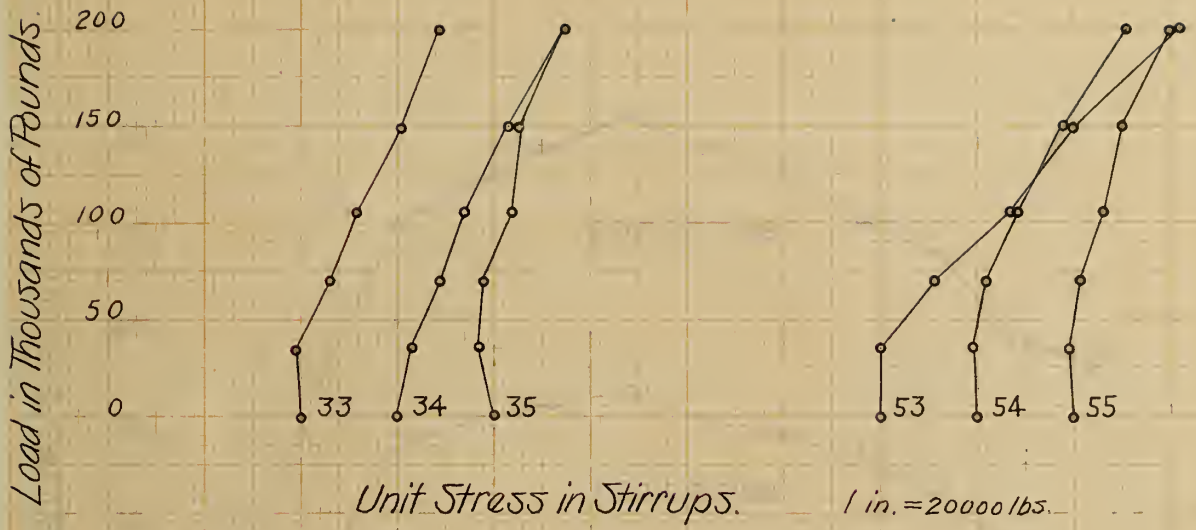




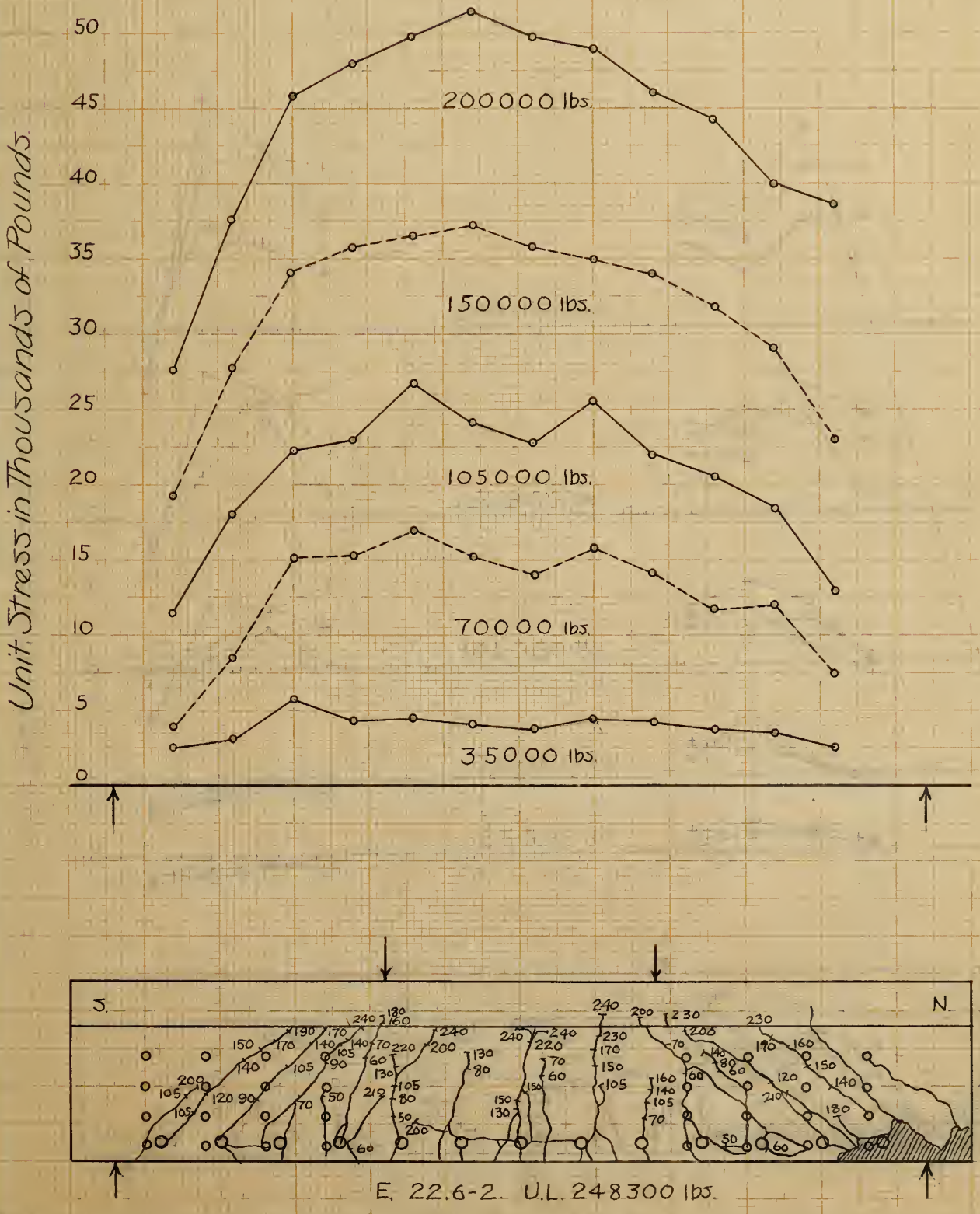
Unit Stress in Stirrups. 1 in. = 20 000 lbs.

22.6-2 W.

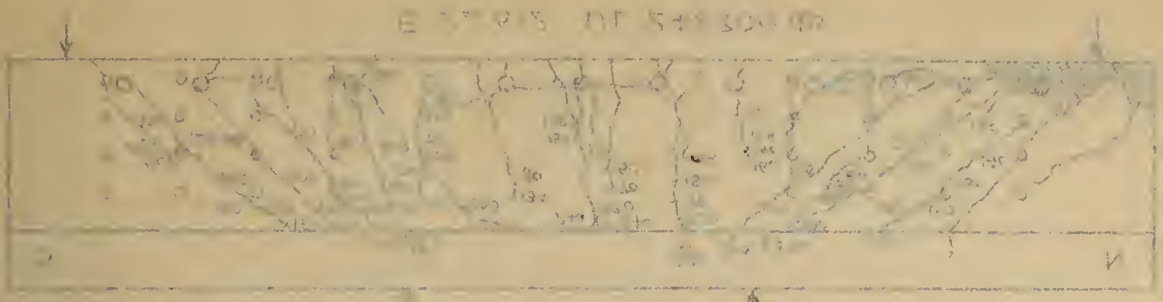


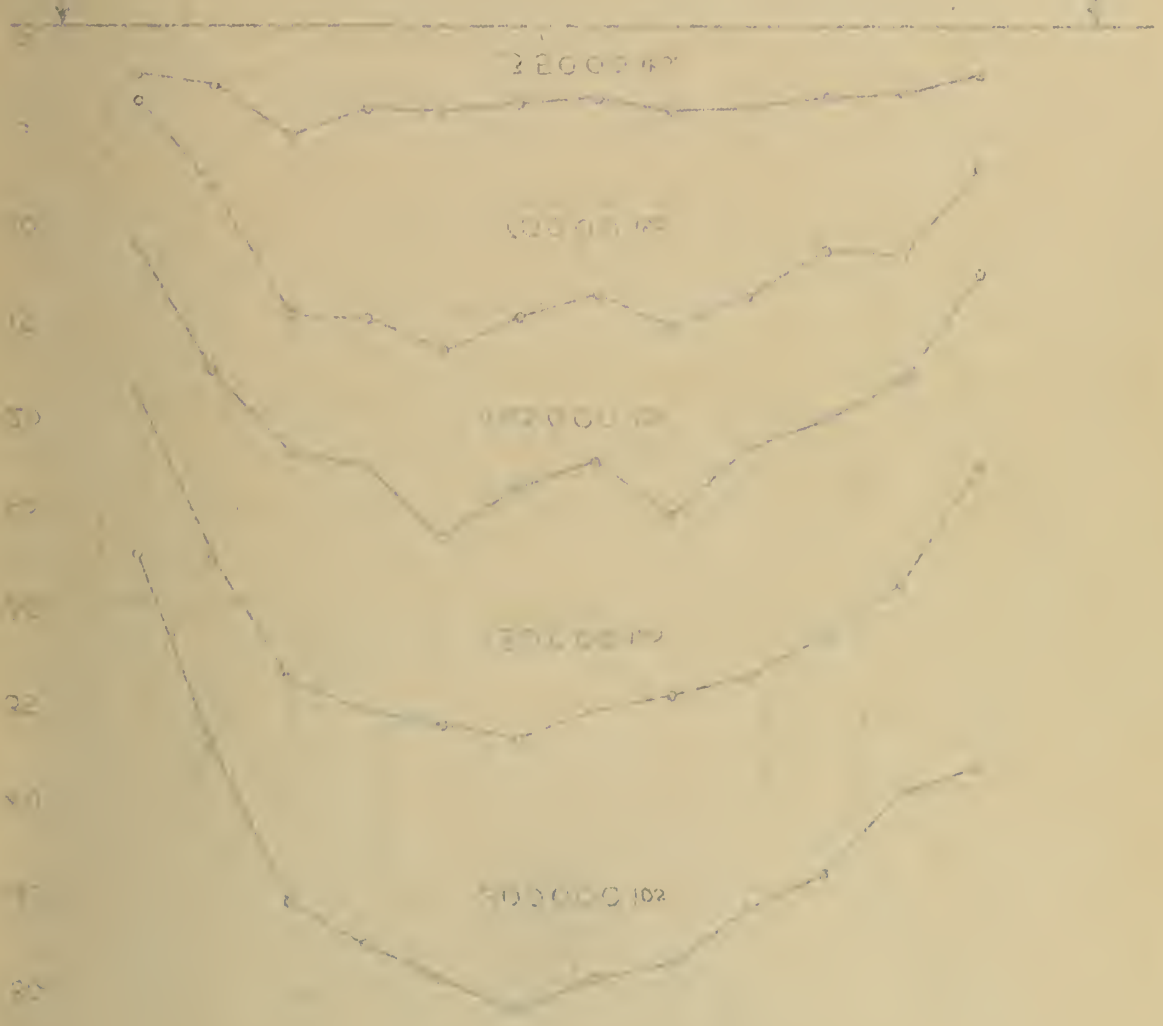


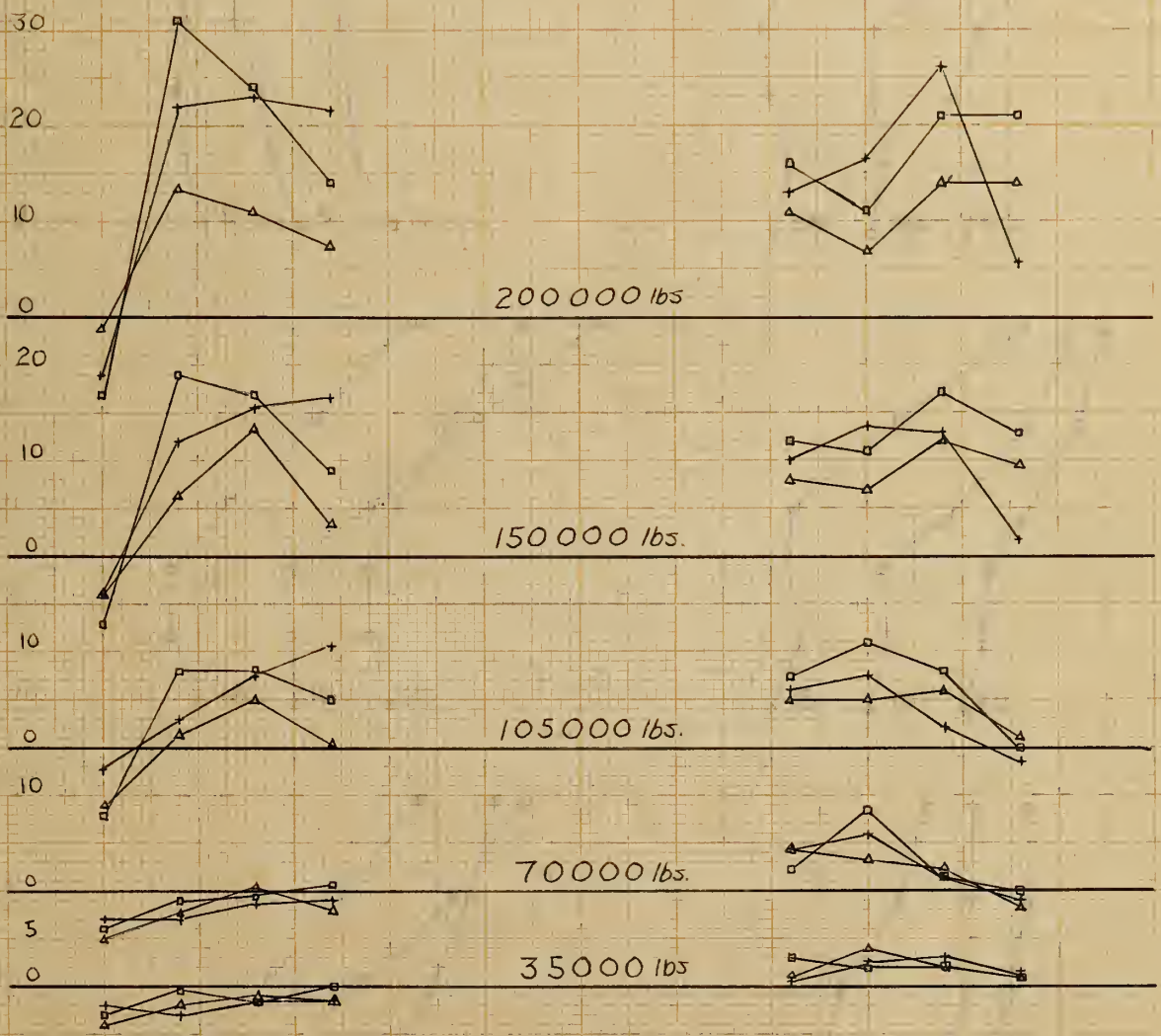
Unit Stress in Stirrups. 1 in. = 20000 lbs.

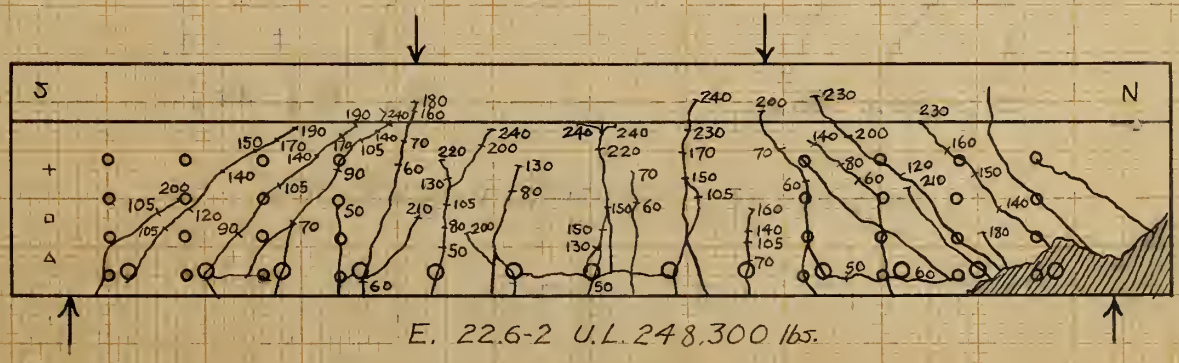


E. 22.6-2. U.L. 248300 lbs.

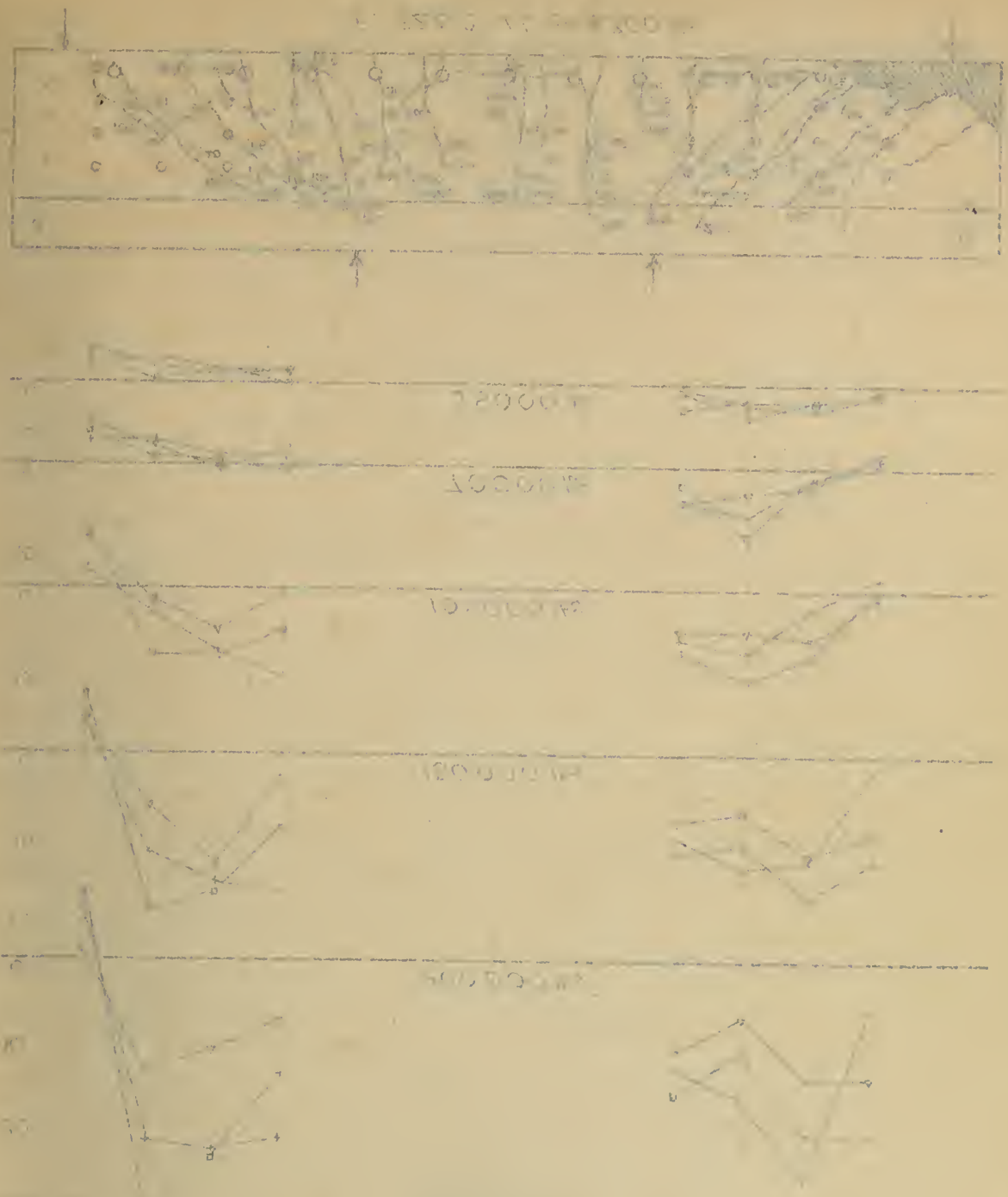


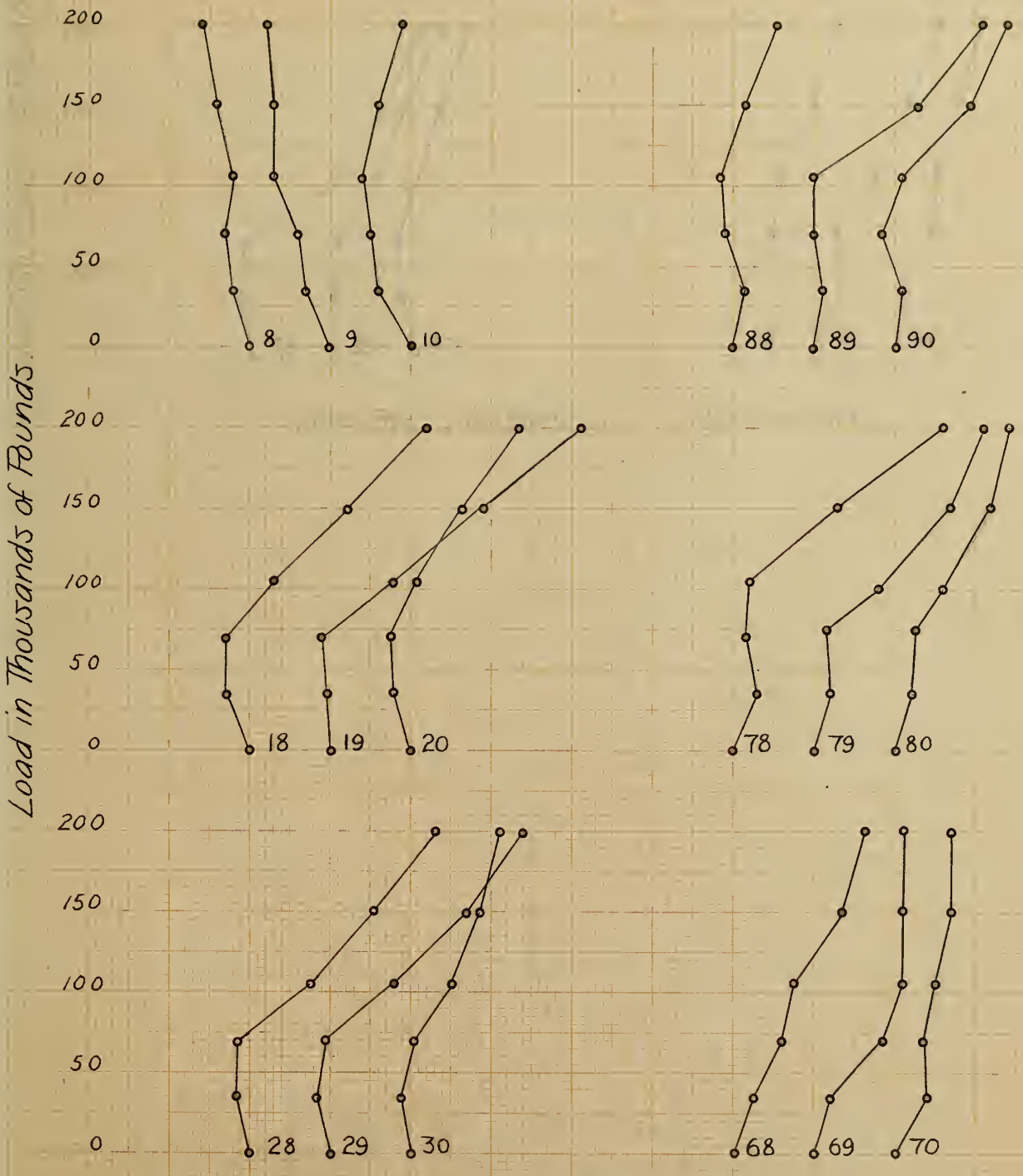




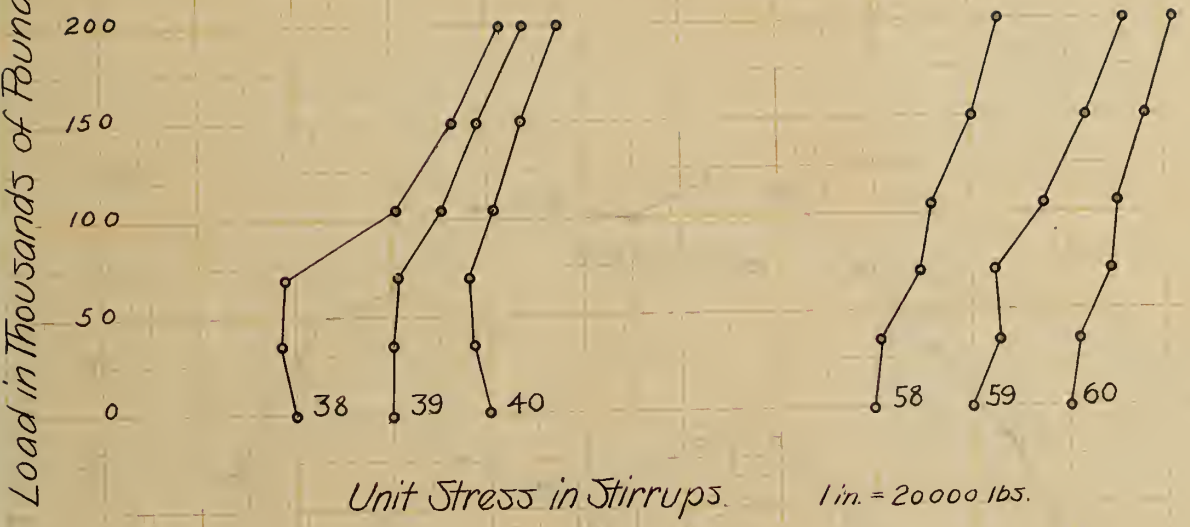


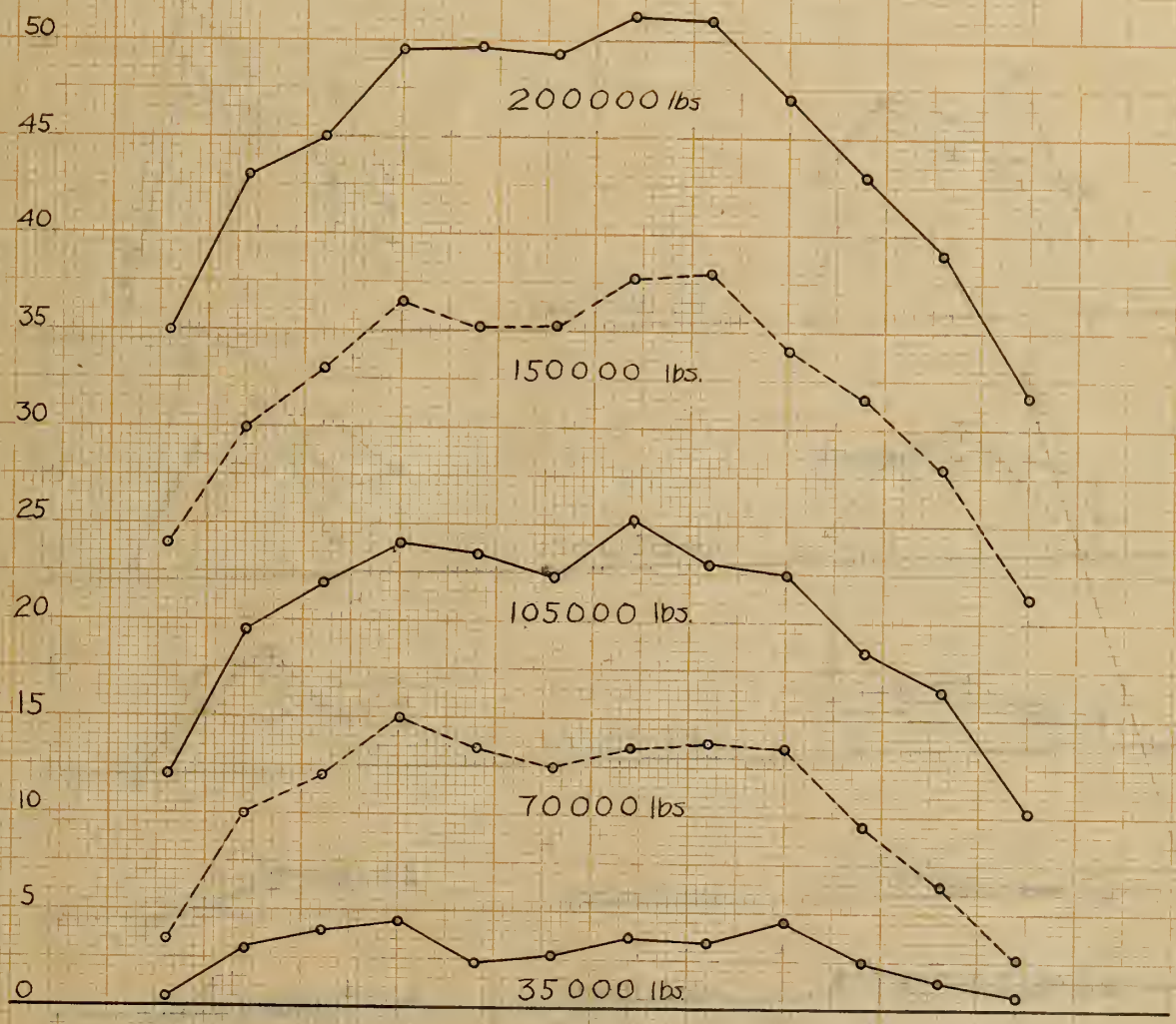
E. 22.6-2 U.L. 248,300 lbs.

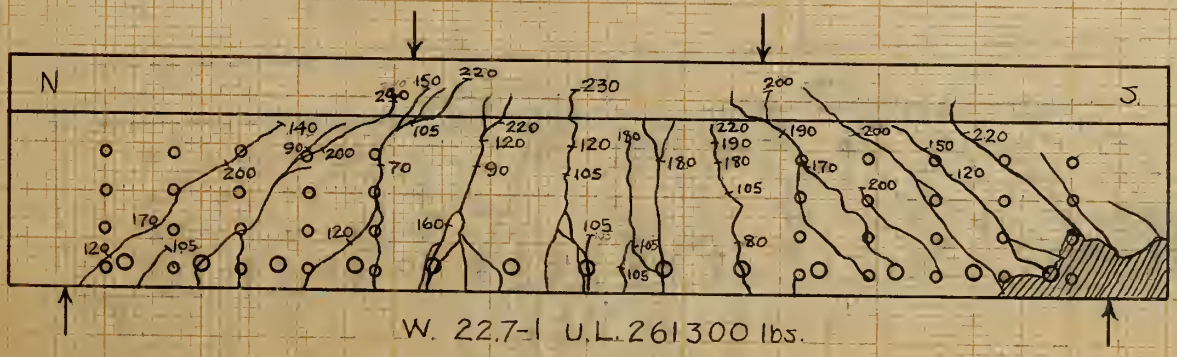


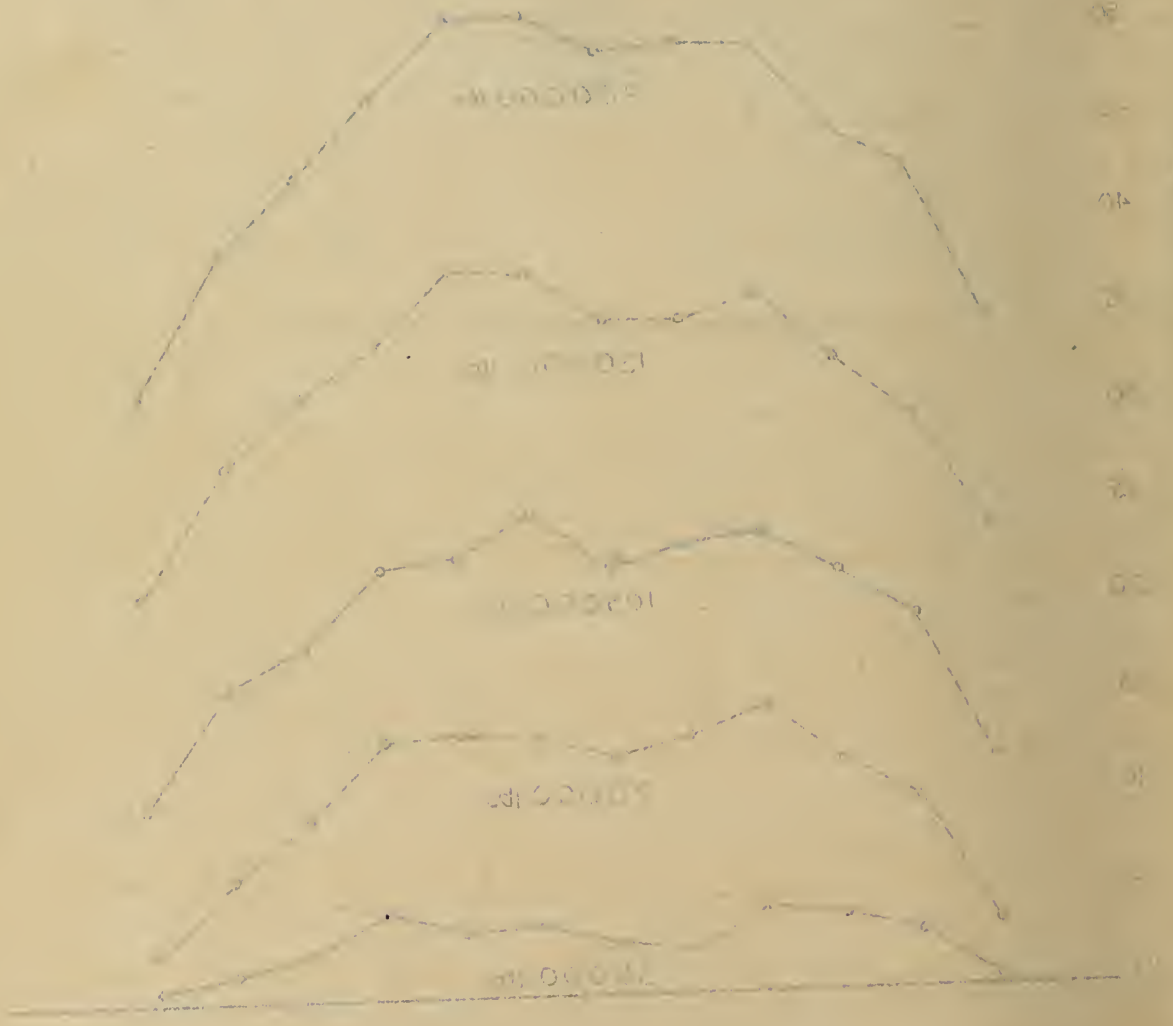

200
150
100
50
0
8
9
10
88
89
90
18
19
20
78
79
80
28
29
30
68
69
70
Load in Thousands of Pounds.
Unit Stress in Stirrups
1 in. = 20000 lbs.

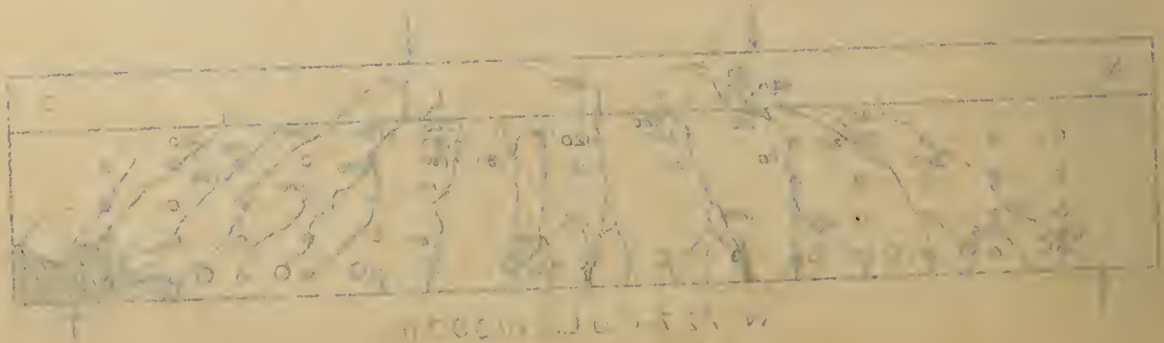
22,6-2. E.

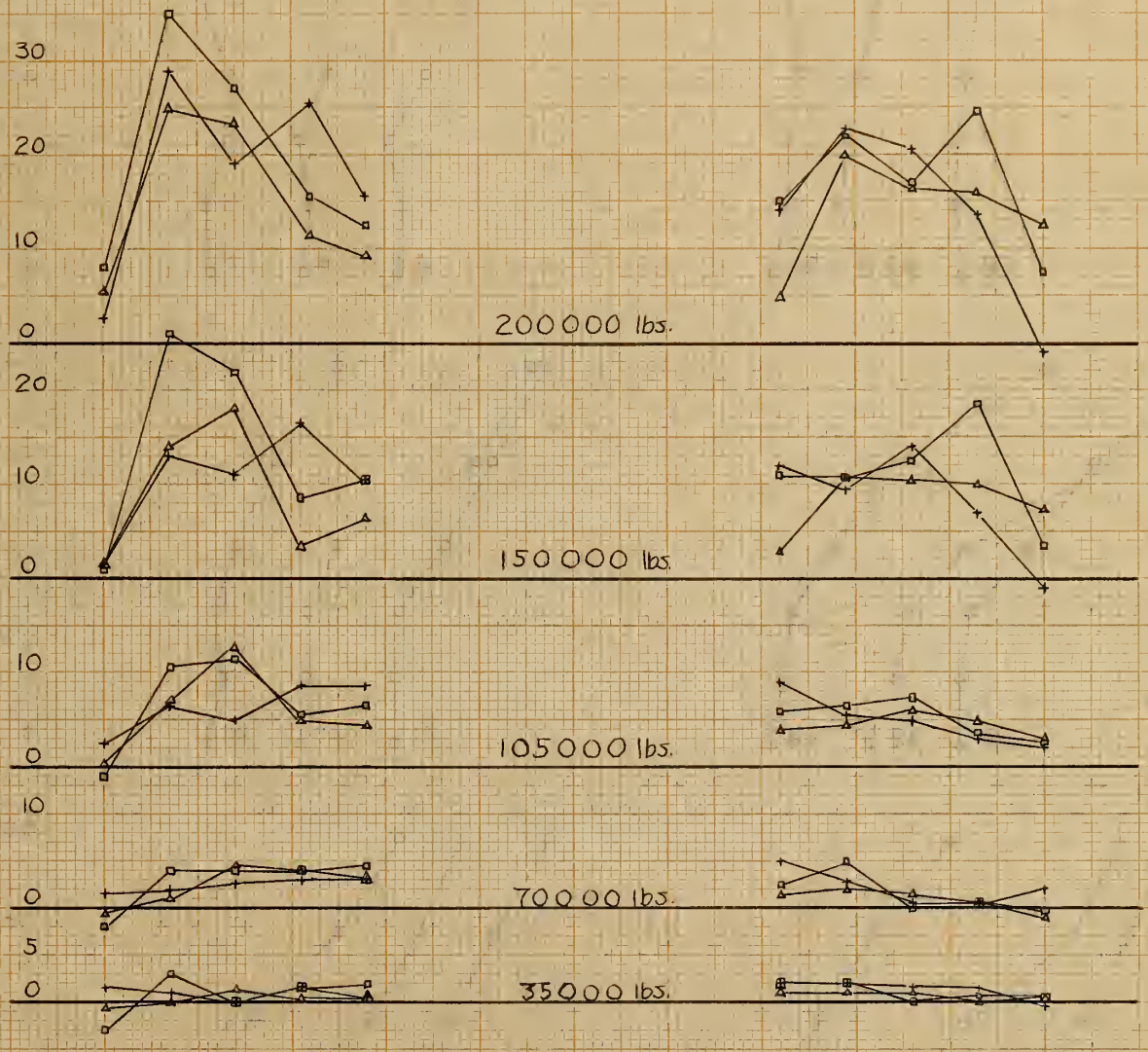

Load in Thousands of Pounds.
200
150
100
50
0
38
39
40
58
59
60
Unit Stress in Stirrups.
1 in. = 20000 lbs.

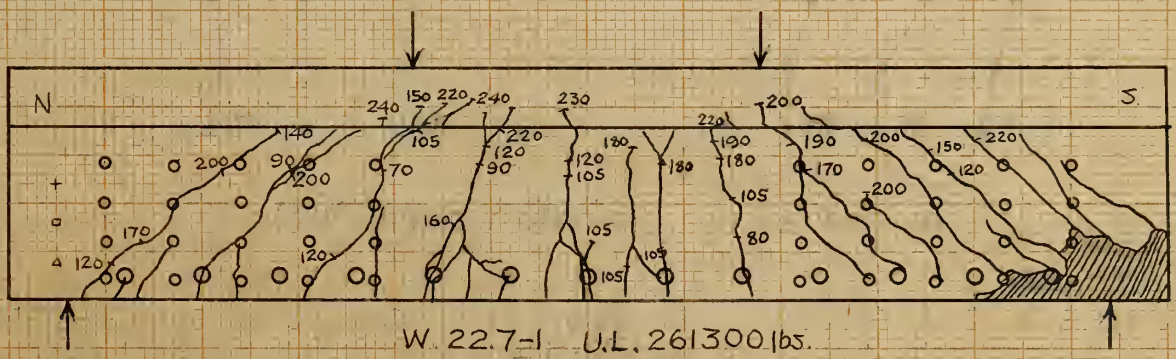

Unit Stress in Thousands of Pounds
50
45
40
35
30
25
20
15
10
5
0
200000 lbs
150000 lbs.
105000 lbs.
70000 lbs
35000 lbs

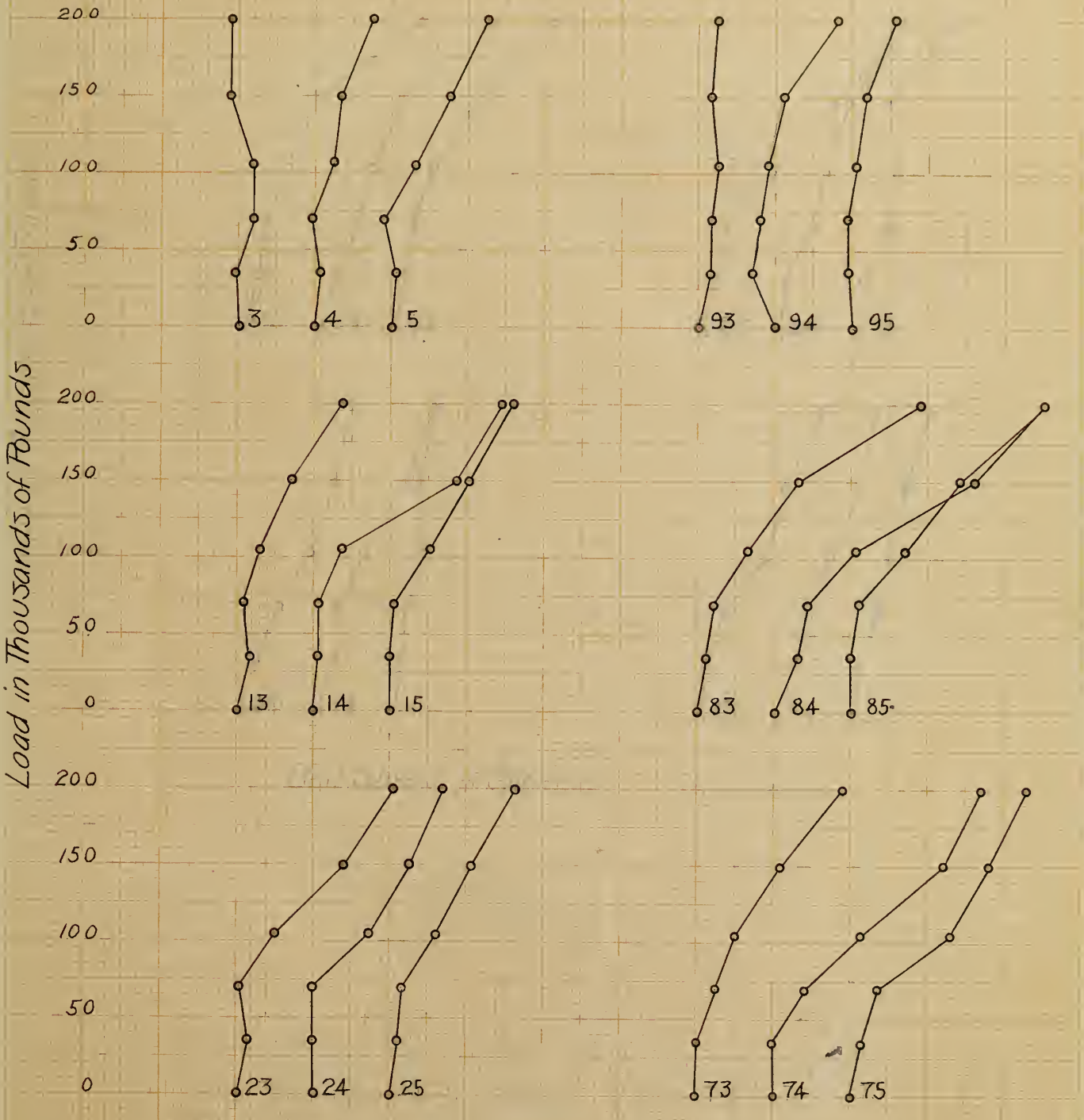

N
S.
W. 22.7-1 U.L. 261300 lbs.



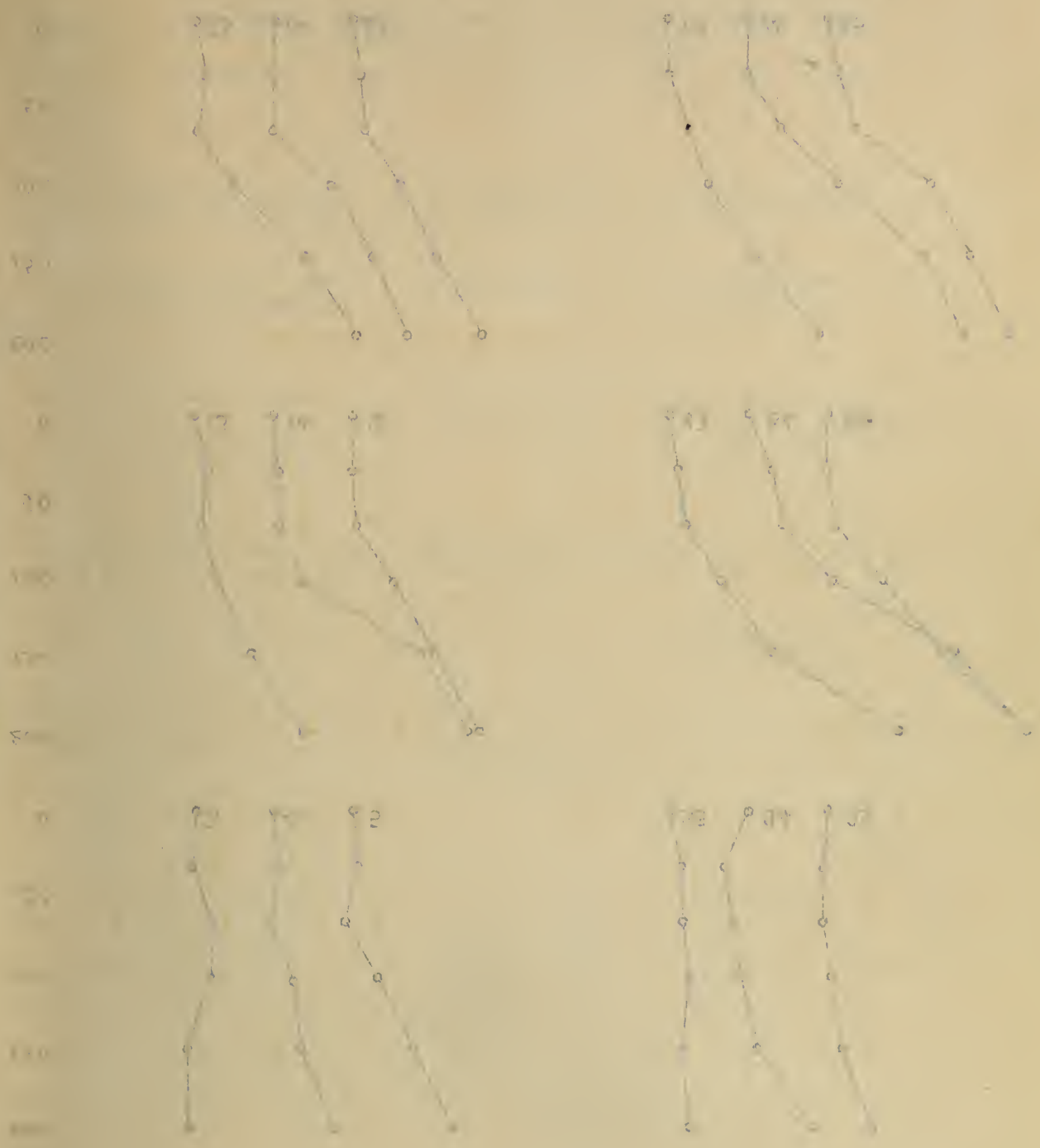


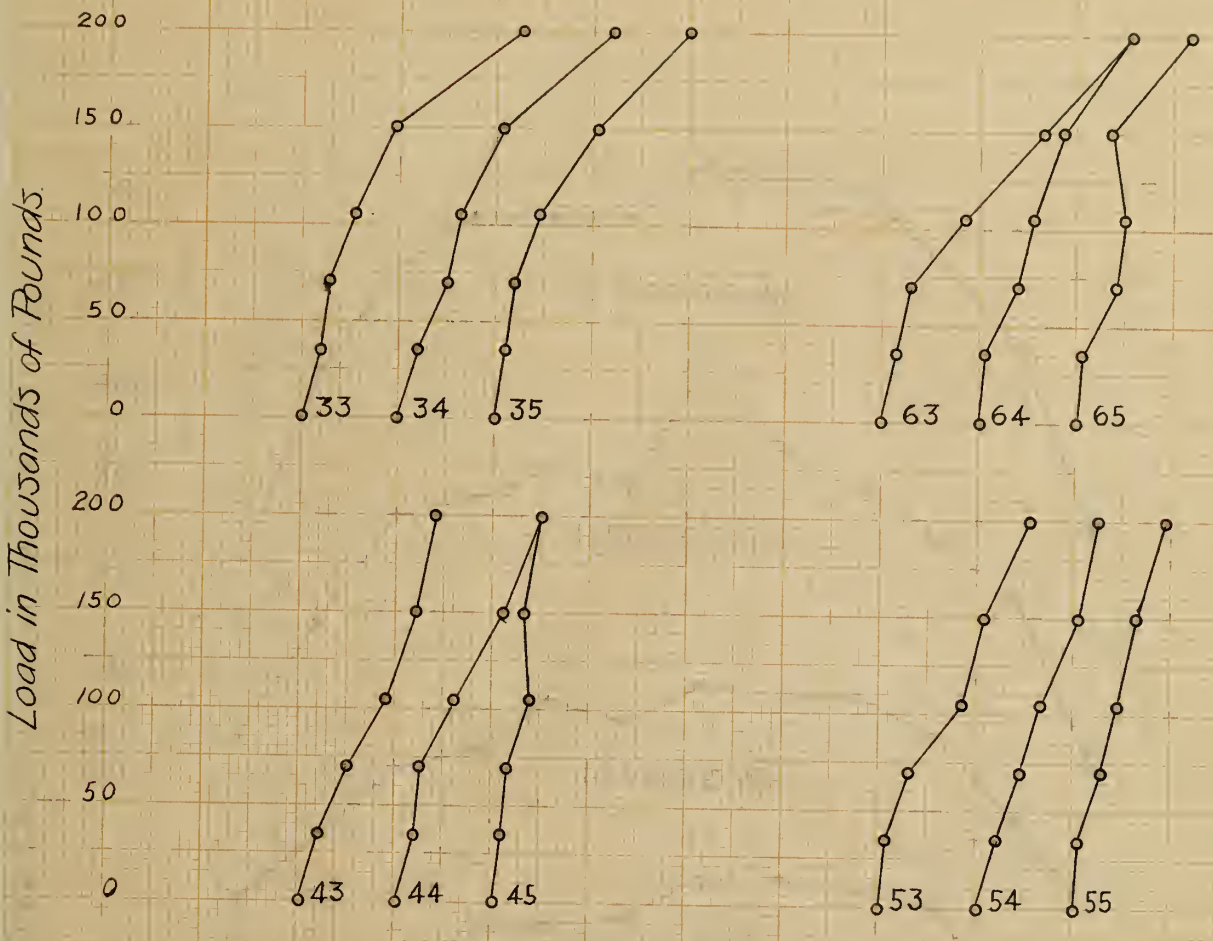

Unit Stress in Thousands of Pounds
30
20
10
0
200000 lbs.
20
10
0
150000 lbs.
10
0
105000 lbs.
10
0
70000 lbs.
5
0
35000 lbs.

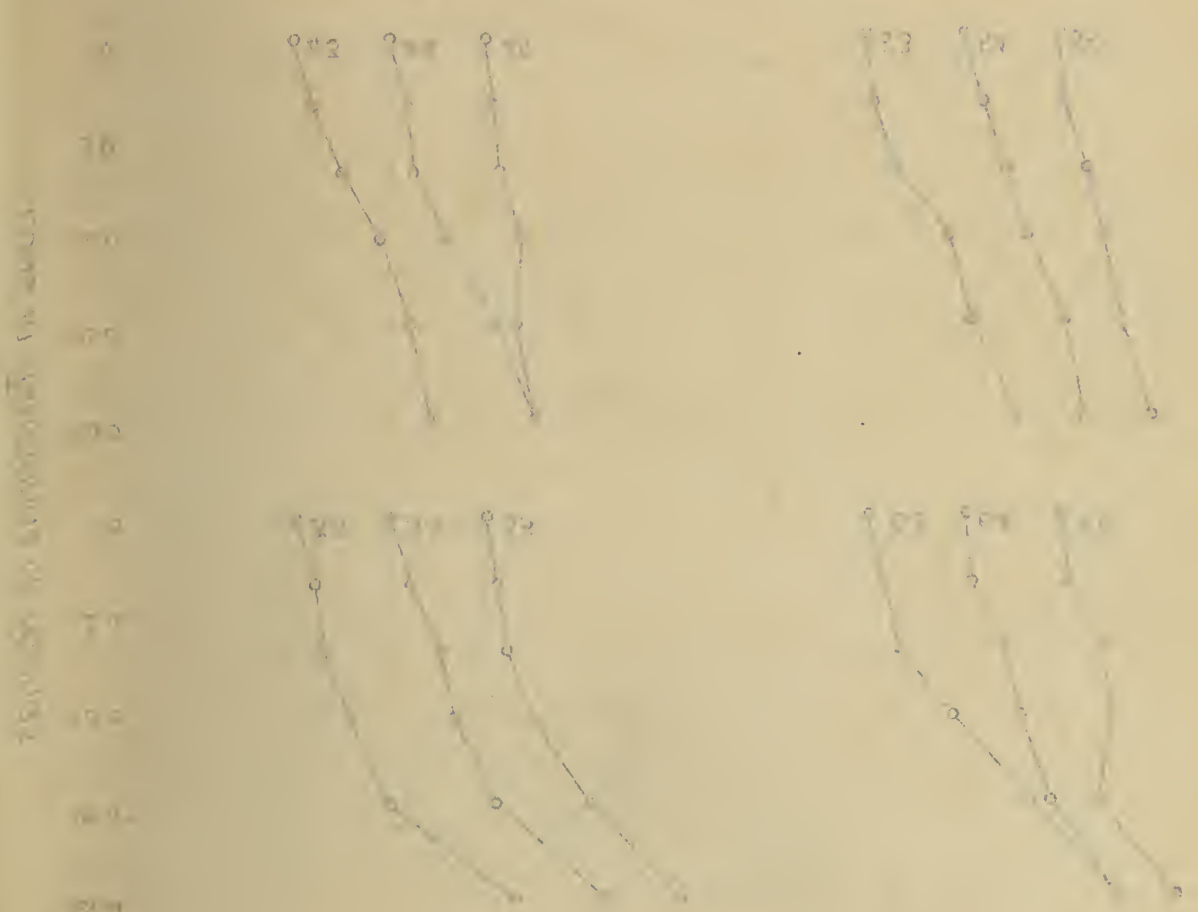

N
S
W. 22.7-1 U.L. 261300 lbs.

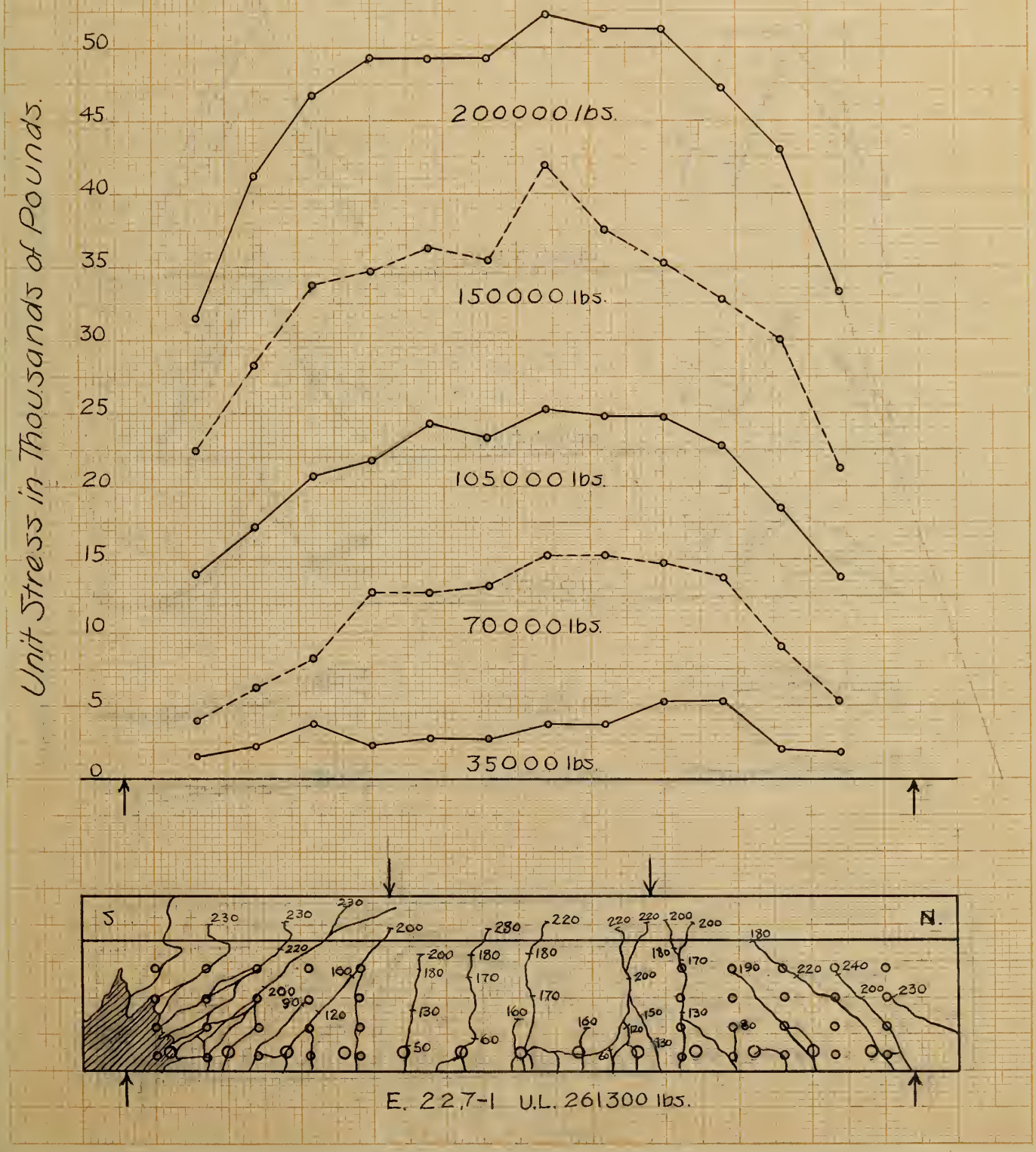

200
150
100
50
0
3
4
5
93
94
95
200
150
100
50
0
13
14
15
83
84
85
200
150
100
50
0
23
24
25
73
74
75
Load in Thousands of Pounds
Unit Stress in Stirrups.
1 in. = 20000 lbs.

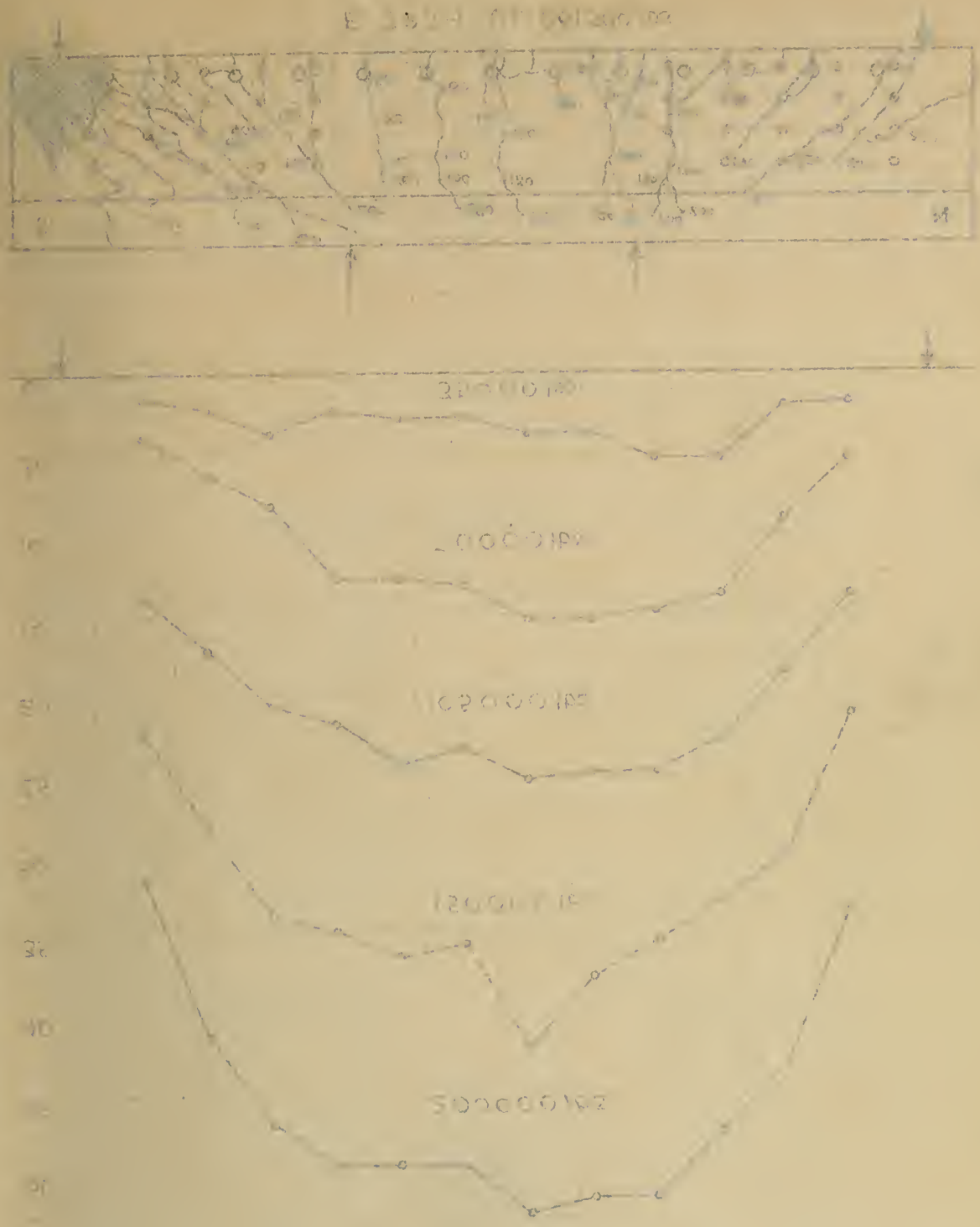
22-7-1 W

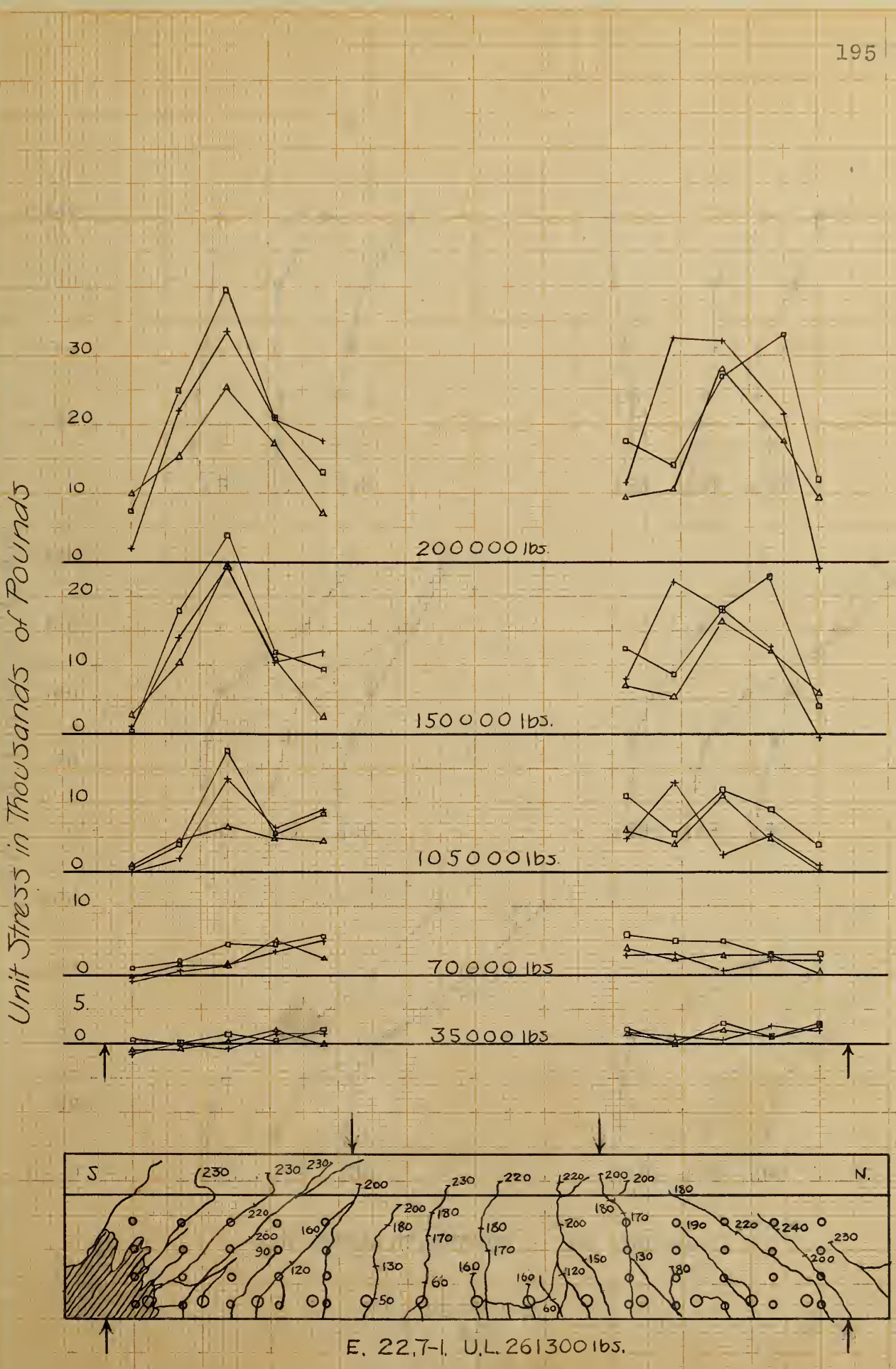


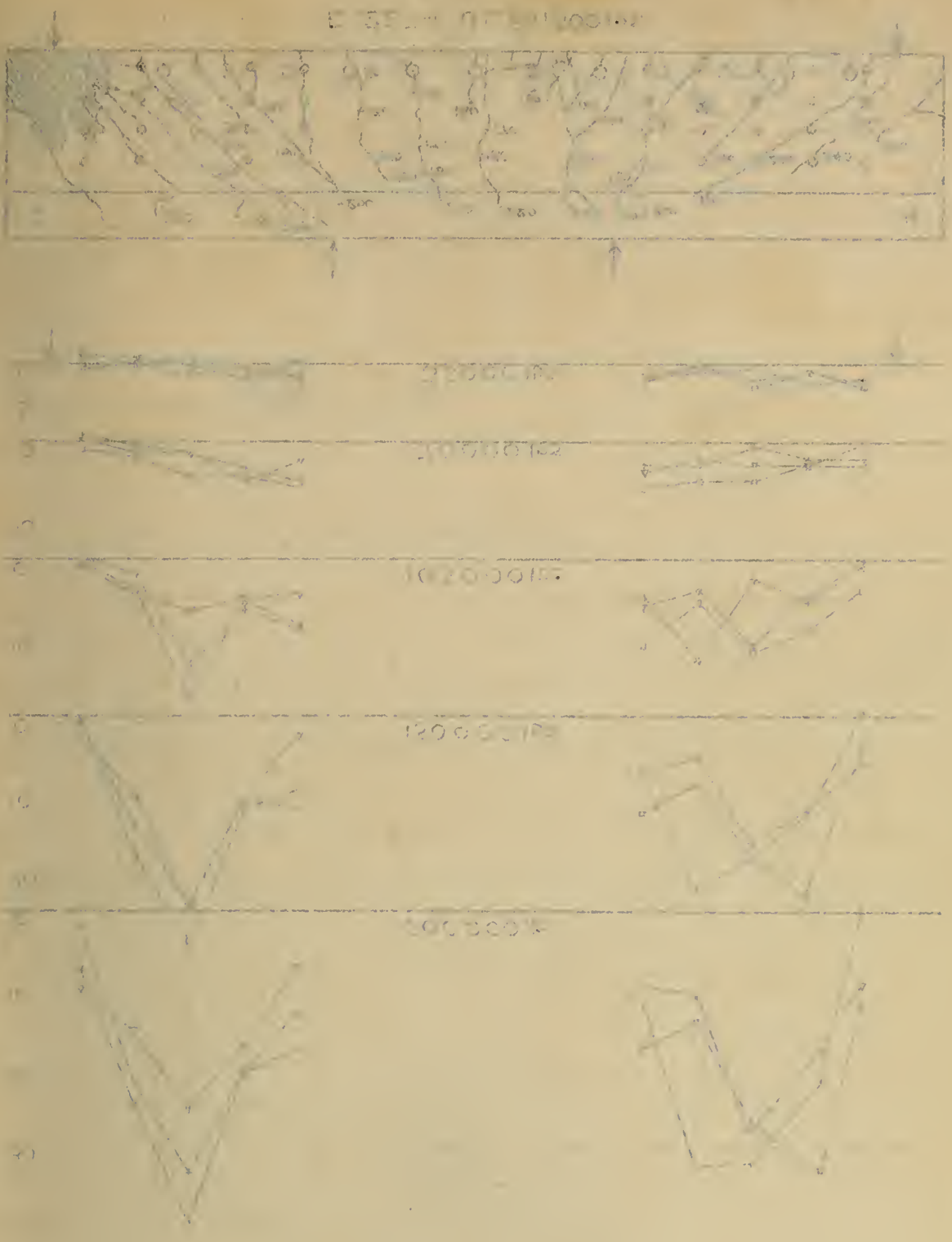

200
150
100
50
0
Load in Thousands of Pounds.
33
34
35
63
64
65
200
150
100
50
0
43
44
45
53
54
55
Unit Stress in Stirrups.
1 in. = 20000 lbs.

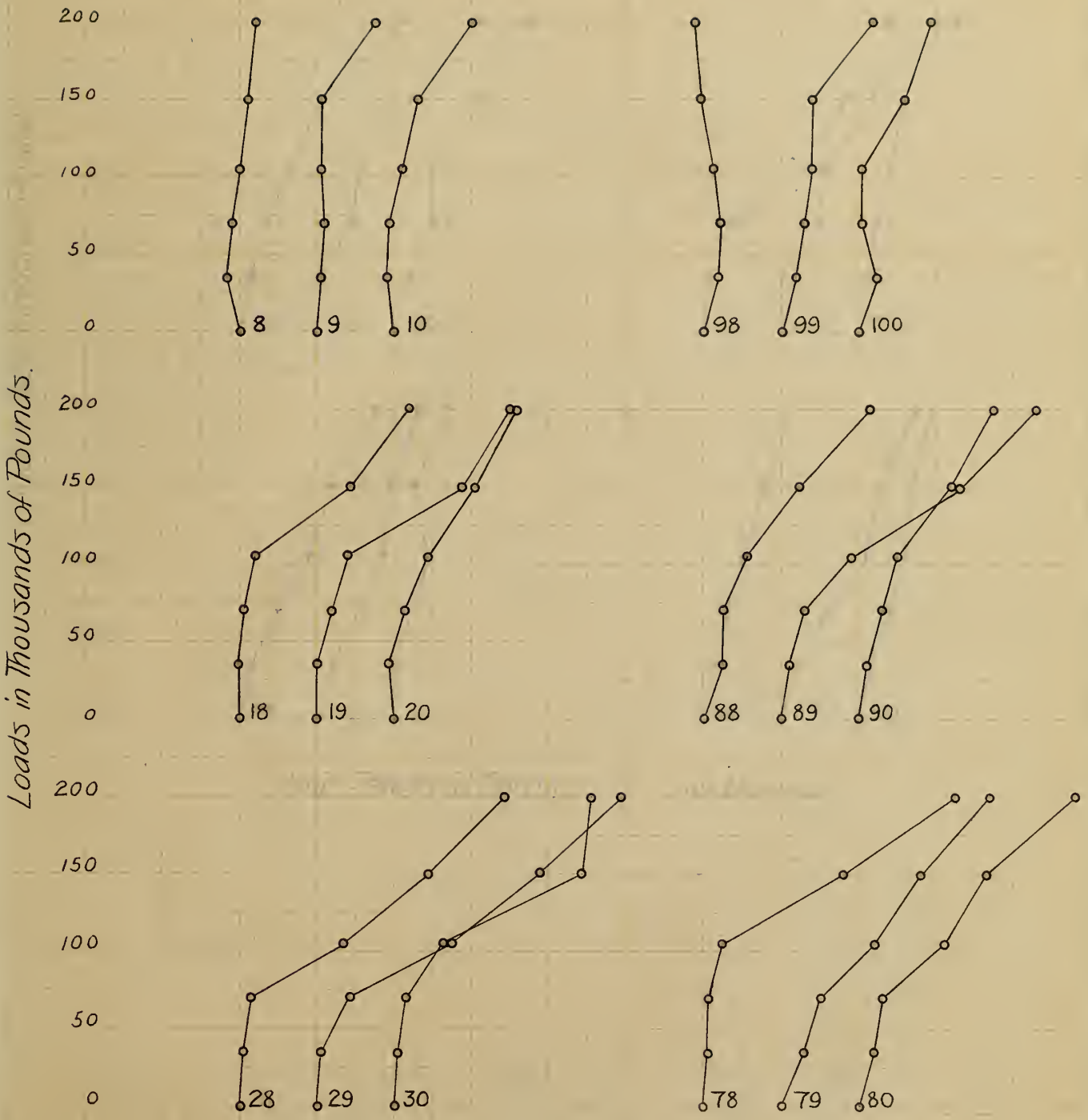



Unit Stress in Thousands of Pounds.
50
45
40
35
30
25
20
15
10
5
0
200000 lbs.
150000 lbs.
105000 lbs.
70000 lbs.
35000 lbs.
S.
N.
E. 22,7-1 U.L. 261300 lbs.

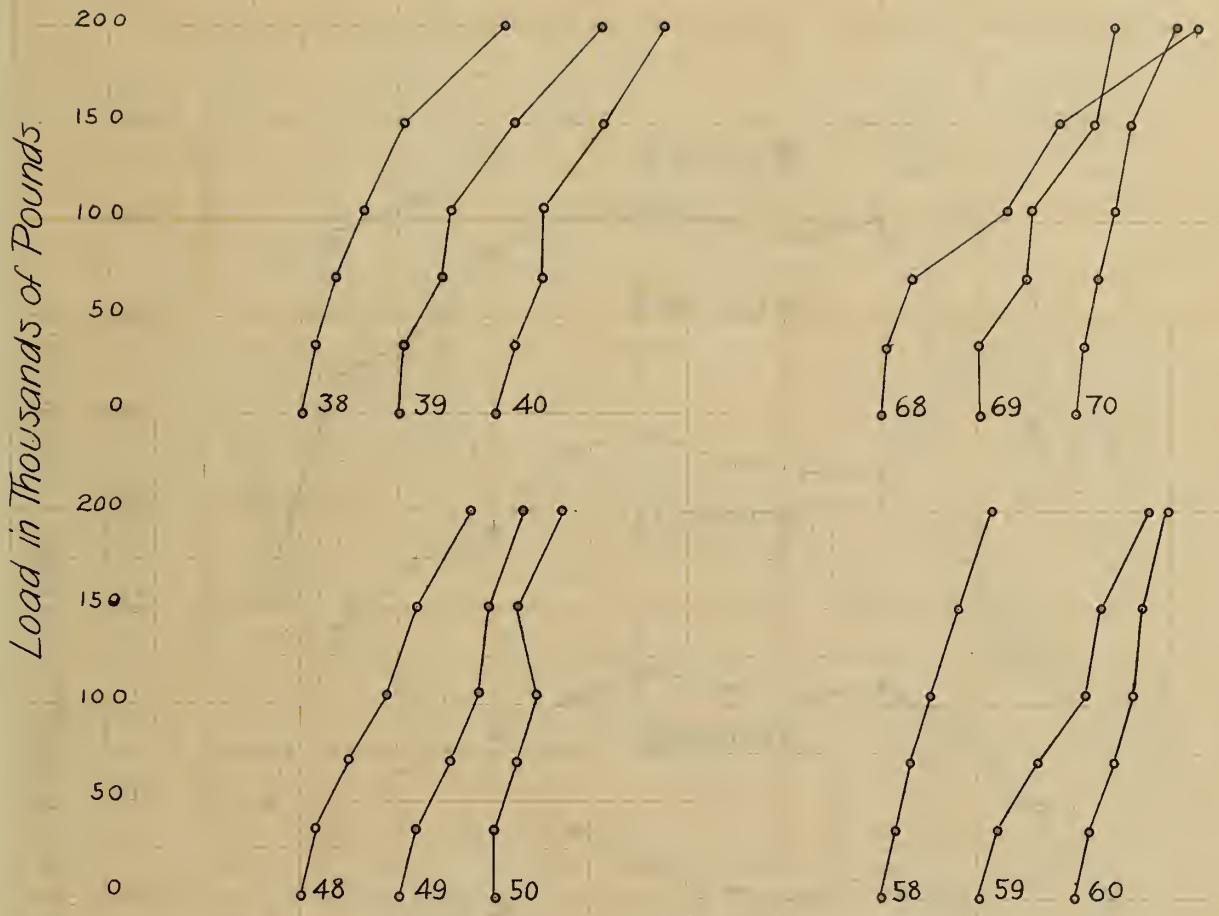



Unit Stress in Thousands of Pounds
30
20
10
0
200000 lbs.
20
10
0
150000 lbs.
10
0
105000 lbs.
10
0
70000 lbs
5.
0
35000 lbs
S
N.
230
230
230
200
220
200
160
90
120
200
180
130
50
230
180
170
60
220
180
170
160
160
60
220
200
200
180
200
170
150
130
120
180
180
190
220
240
230
200
E. 22.7-1. U.L. 261300 lbs.

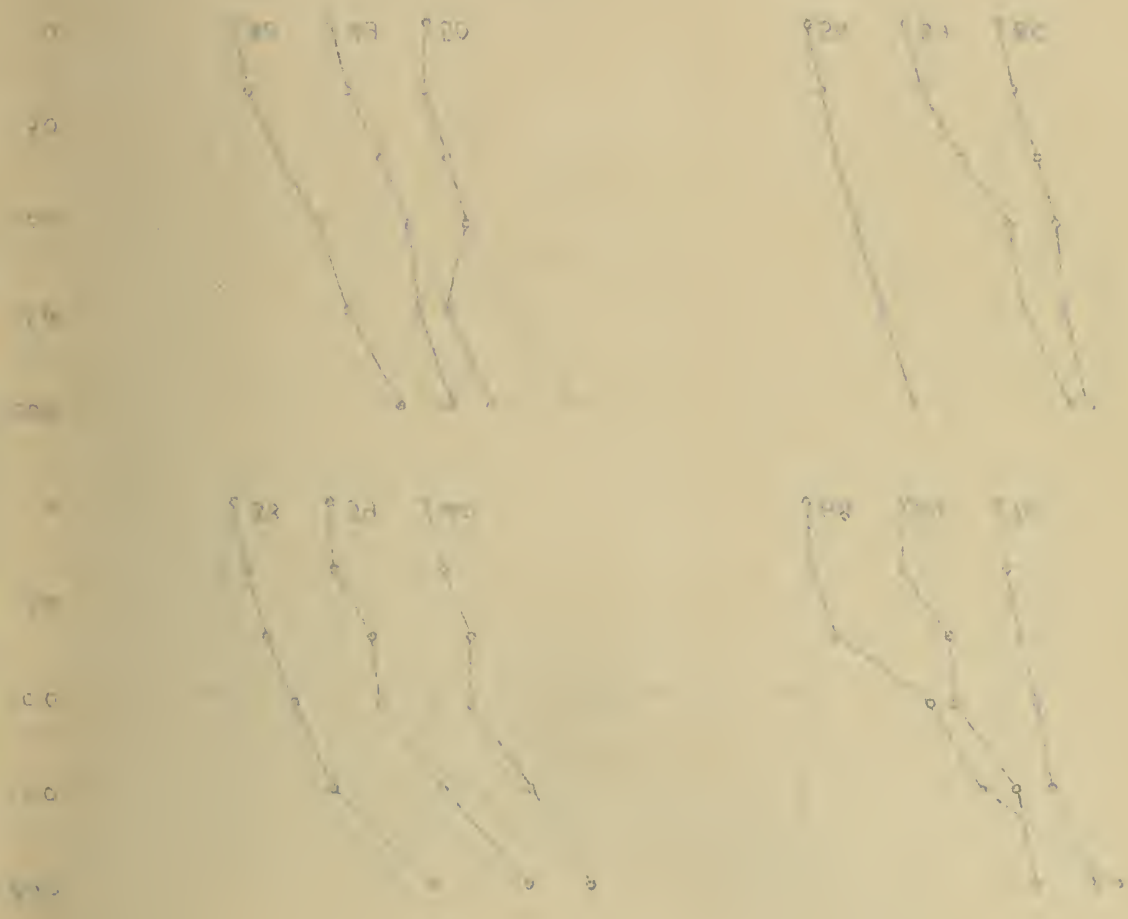


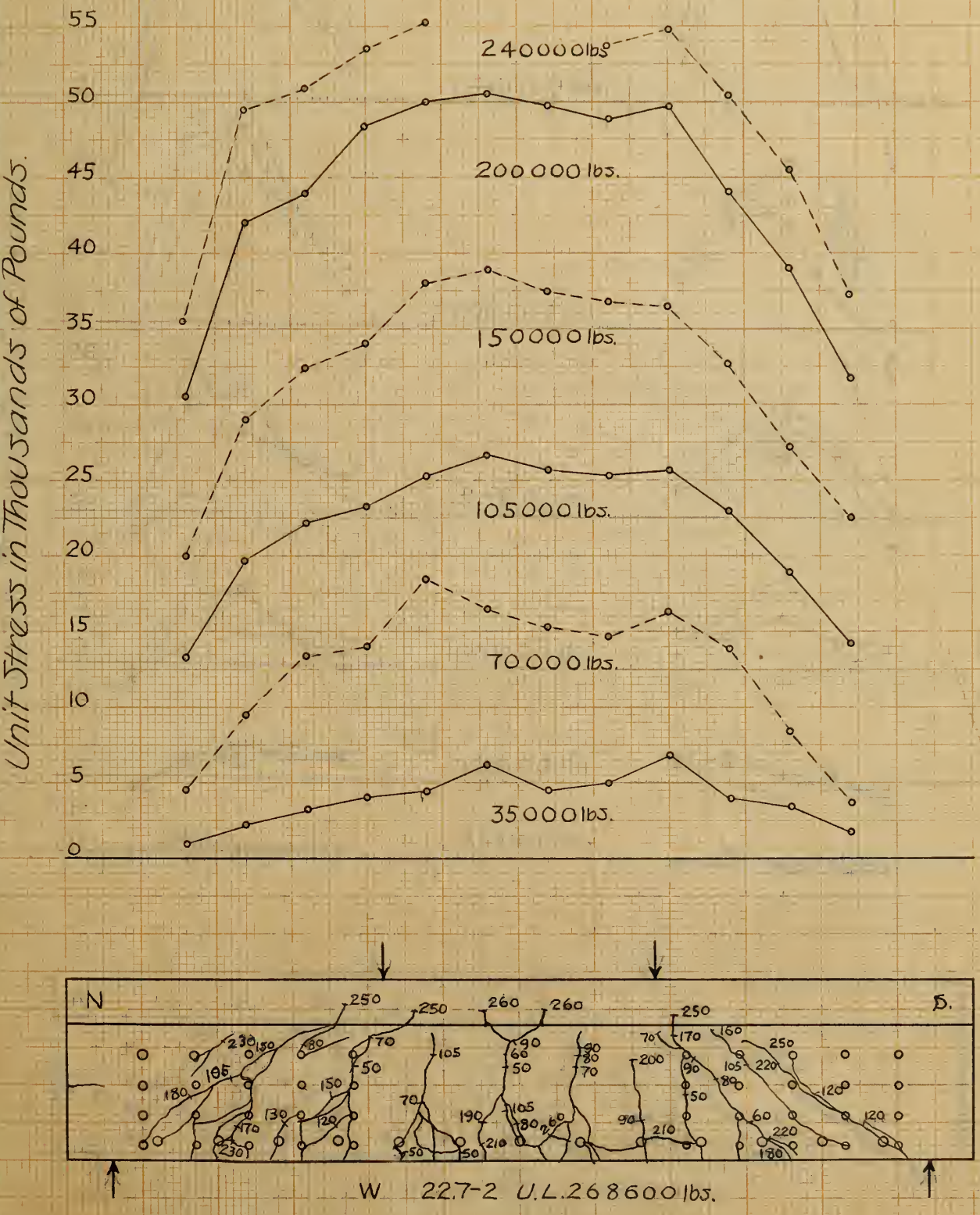

200
150
100
50
0
8
9
10
98
99
100
18
19
20
88
89
90
28
29
30
78
79
80
Loads in Thousands of Pounds.
Unit Stress in Stirrups.
1 in. = 20000 lbs.

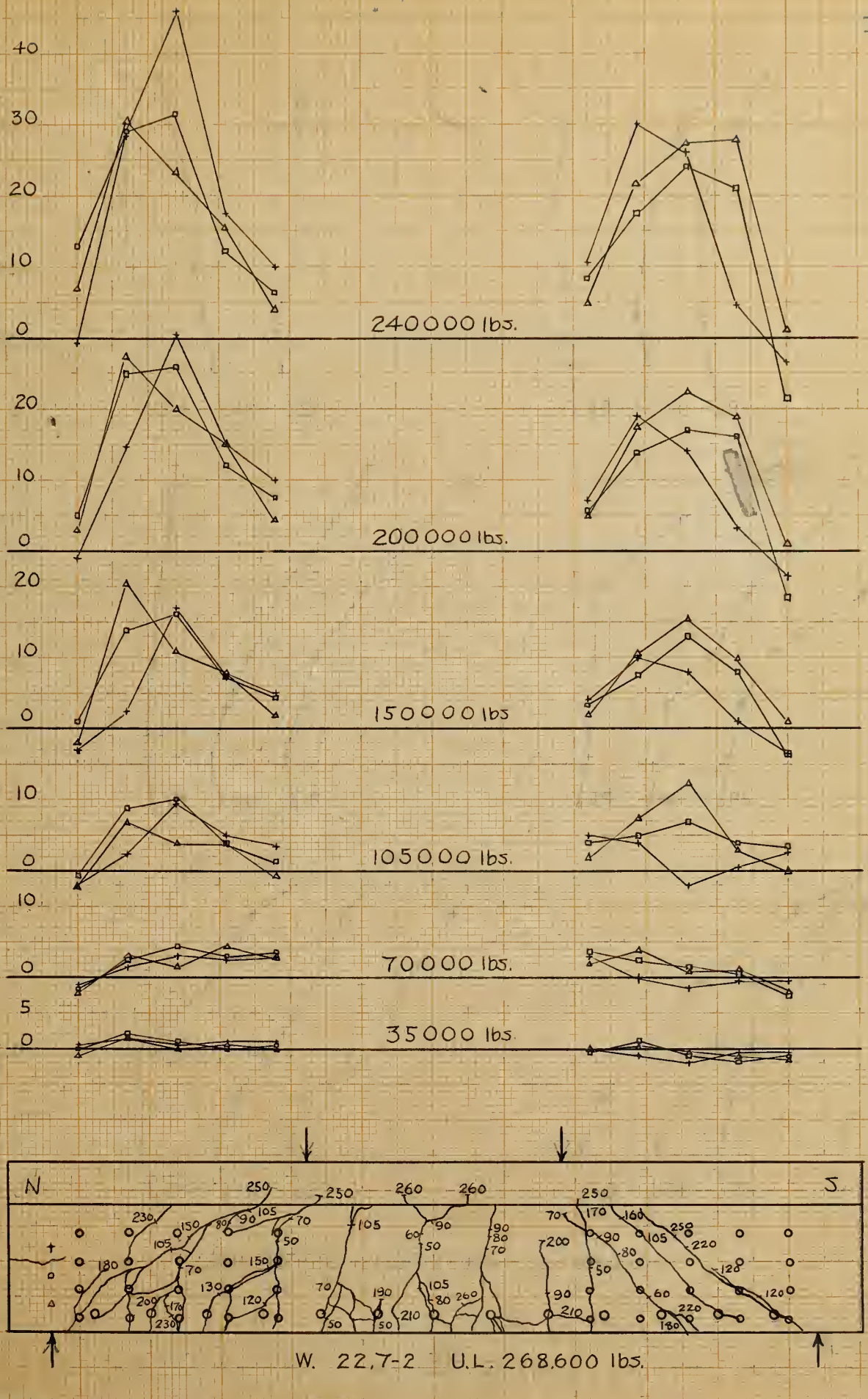
22,7-1 E.

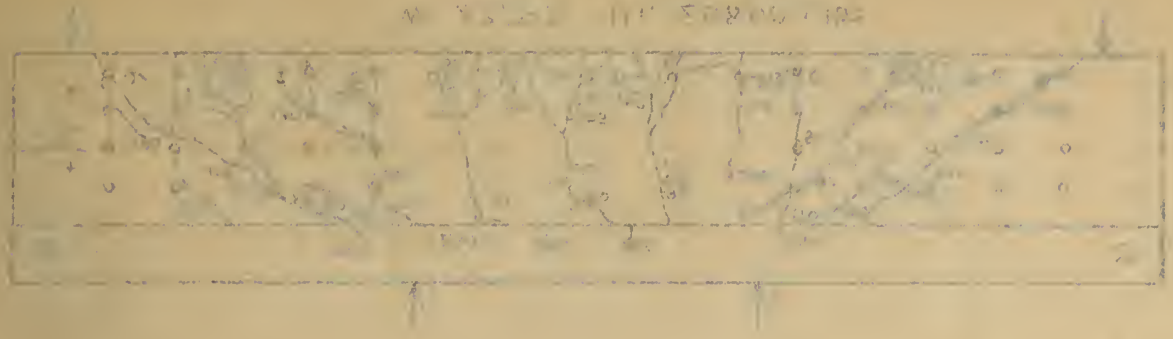


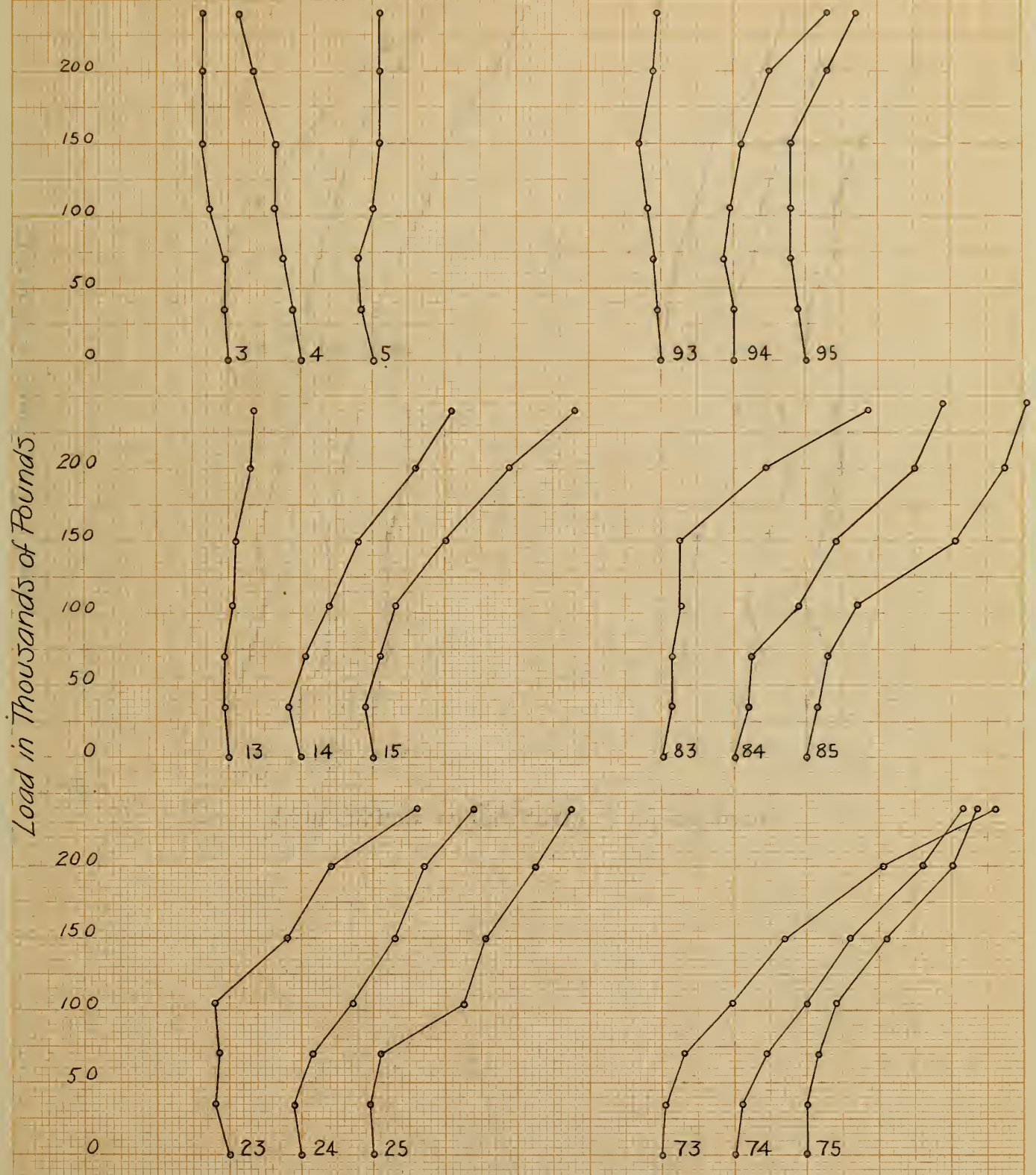
Unit Stress in Stirrups. 1 in. = 20000 lbs.



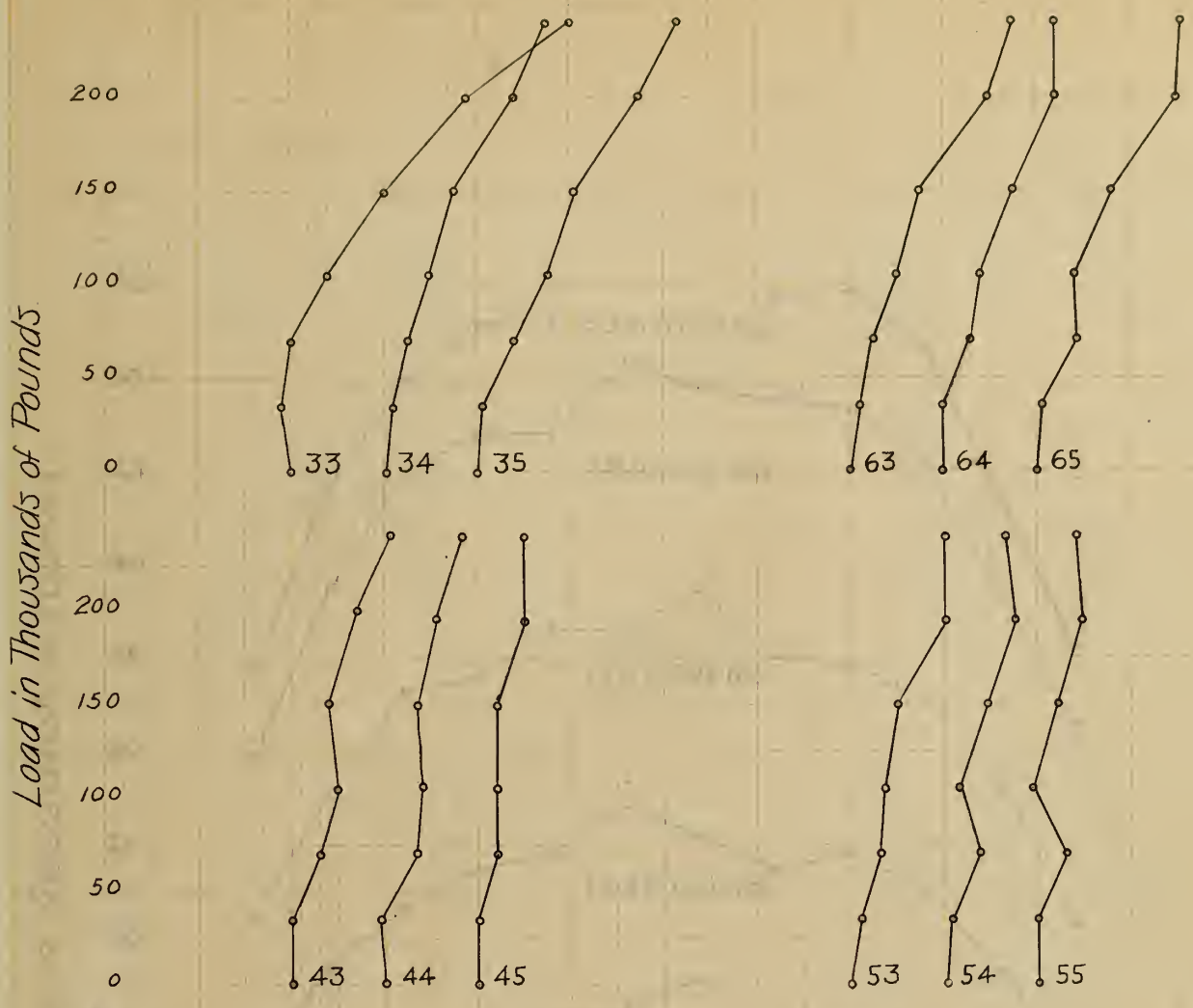

Unit Stress in Thousands of Pounds.
55
50
45
40
35
30
25
20
15
10
5
0
240000 lbs.
200000 lbs.
150000 lbs.
105000 lbs.
70000 lbs.
35000 lbs.
N
S.
W 22.7-2 U.L. 268600 lbs.


Unit Stress in Thousands of Pounds.
240000 lbs.
200000 lbs.
150000 lbs
105000 lbs.
70000 lbs.
35000 lbs.
N
S
W. 22,7-2 U.L. 268,600 lbs.

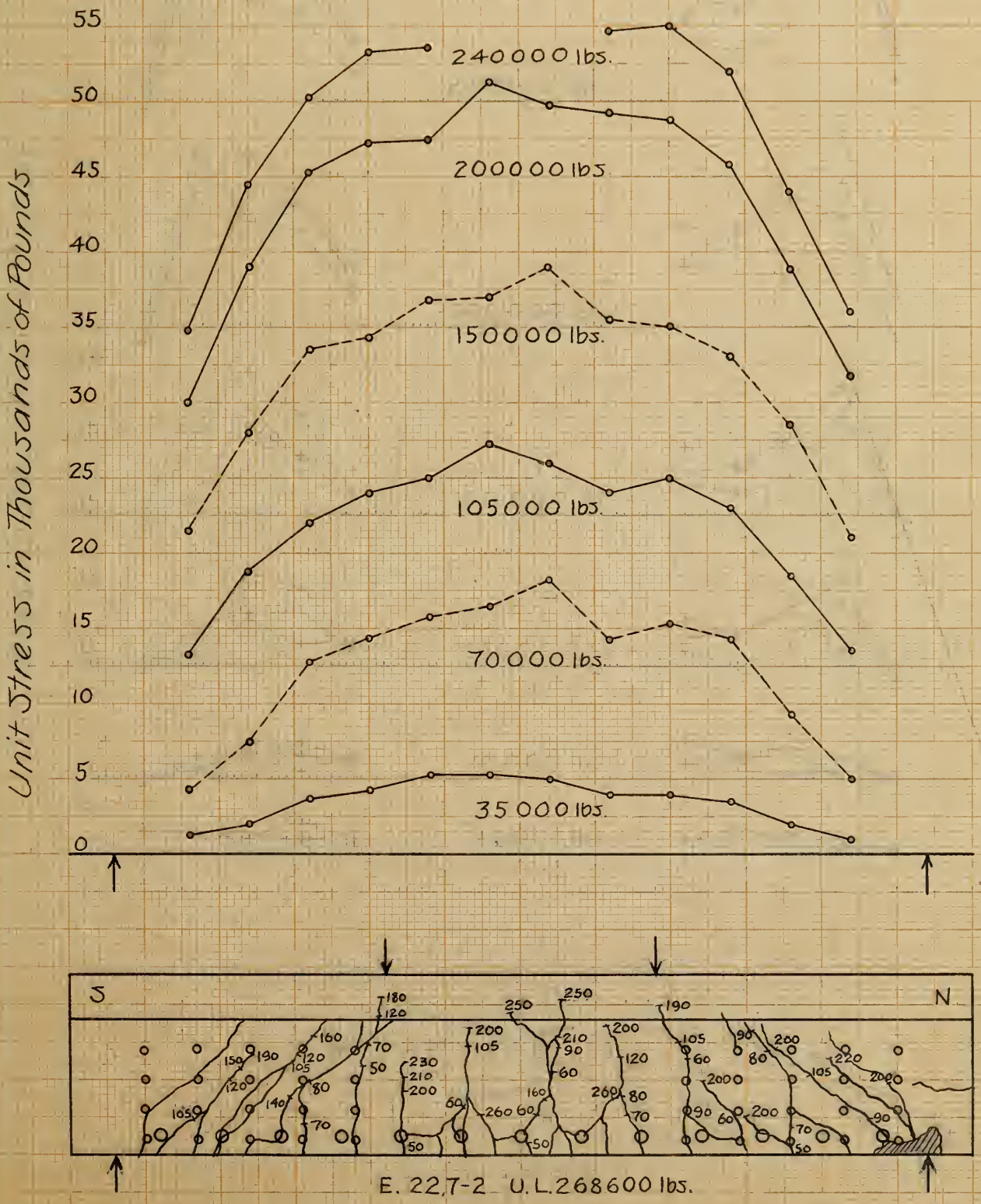


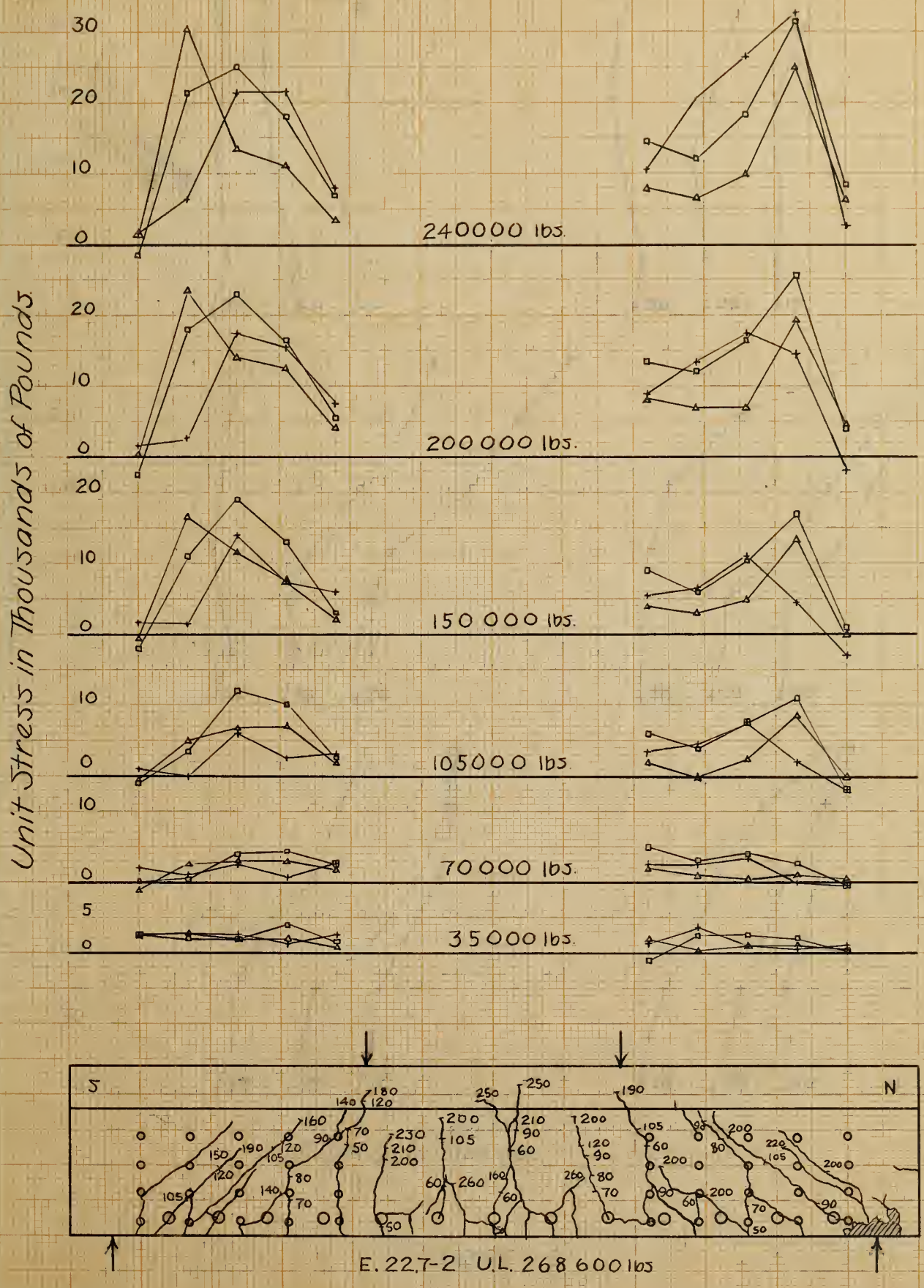


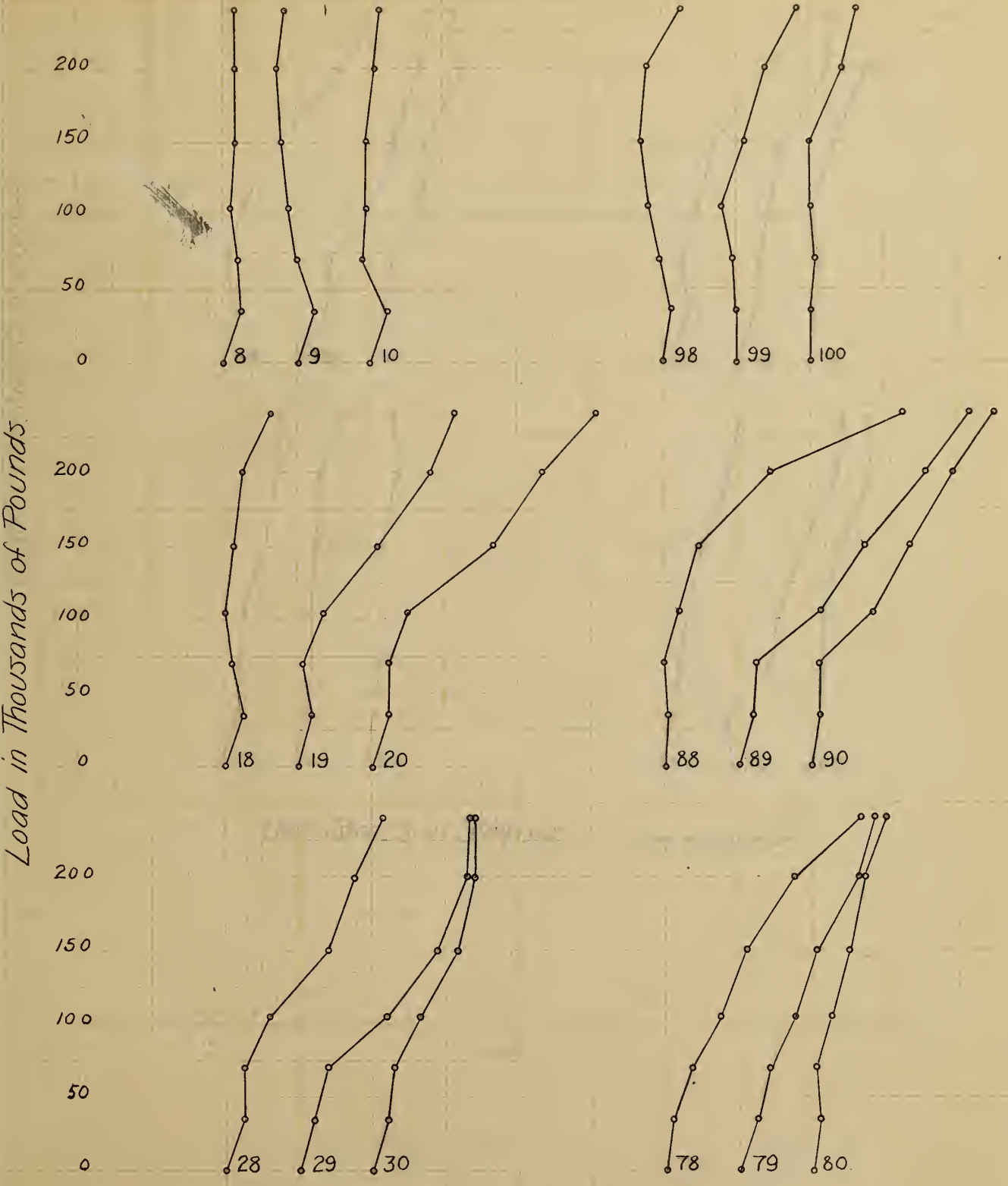
Unit Stress in Stirrups 1 in = 20000 lbs.

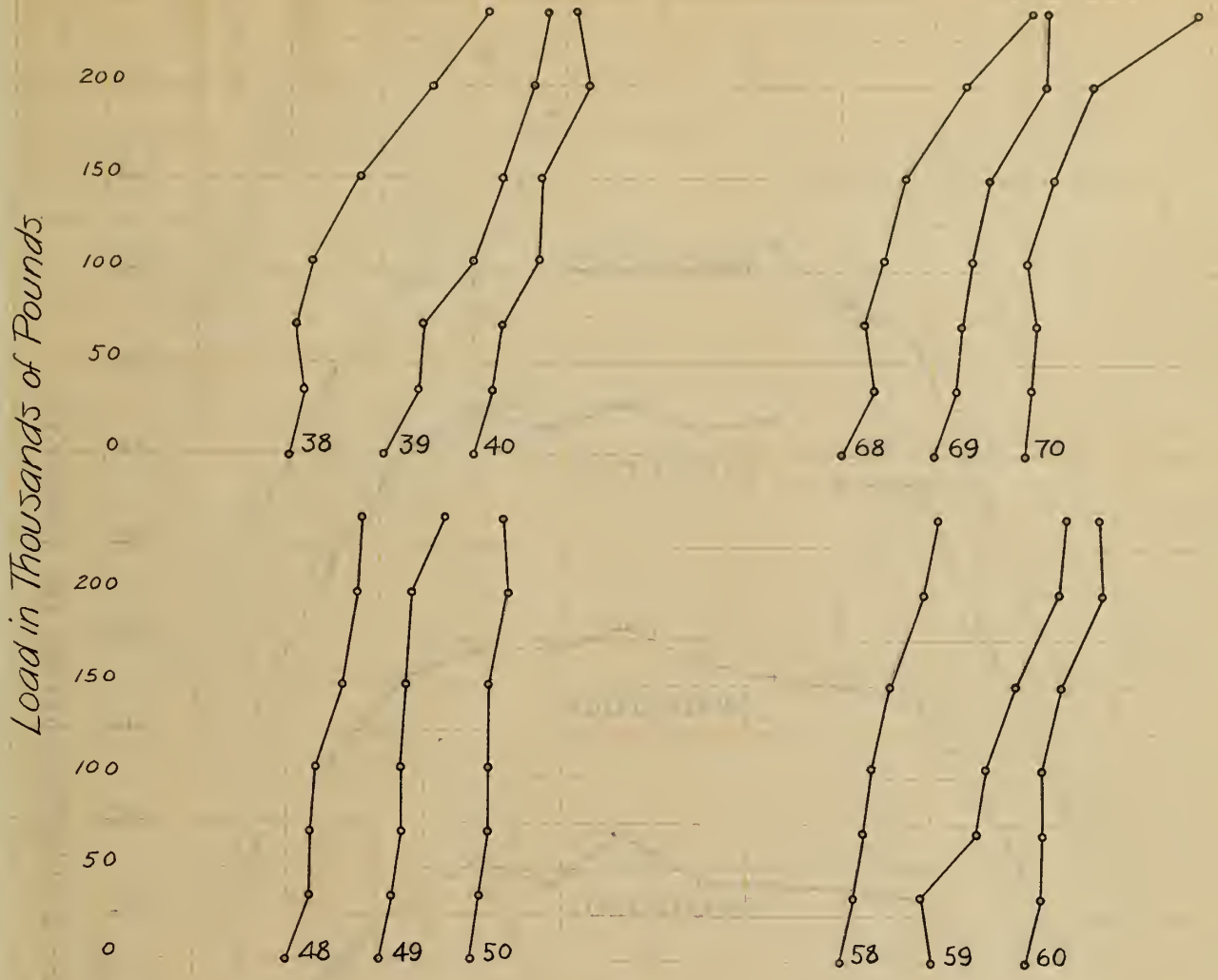


Unit Stress in Stirrups 1 in. = 20 000 lbs.

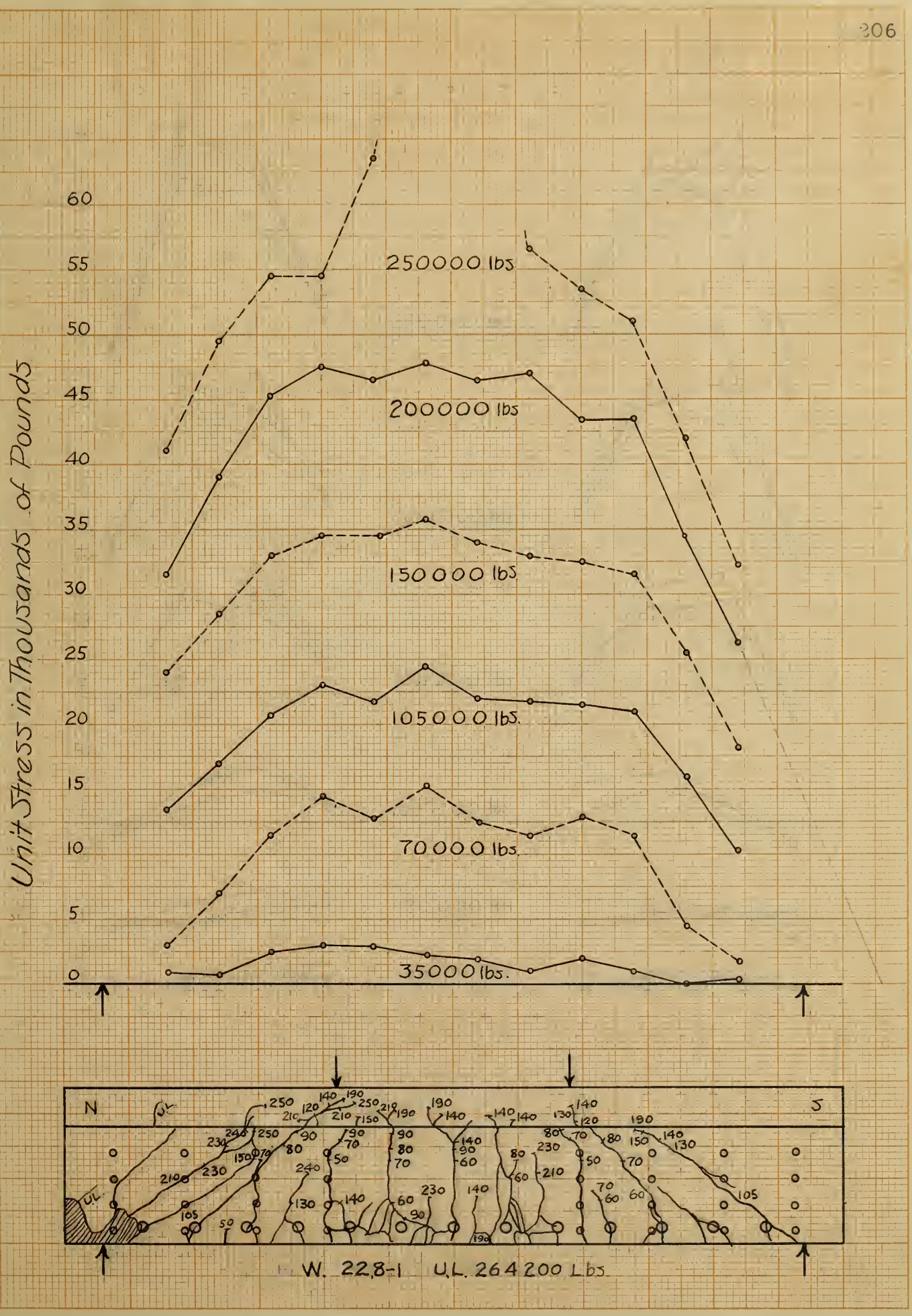

Unit Stress in Thousands of Pounds
55
50
45
40
35
30
25
20
15
10
5
0
240000 lbs.
200000 lbs.
150000 lbs.
105000 lbs.
70000 lbs.
35000 lbs.
S
N
E. 22.7-2 U.L. 268600 lbs.

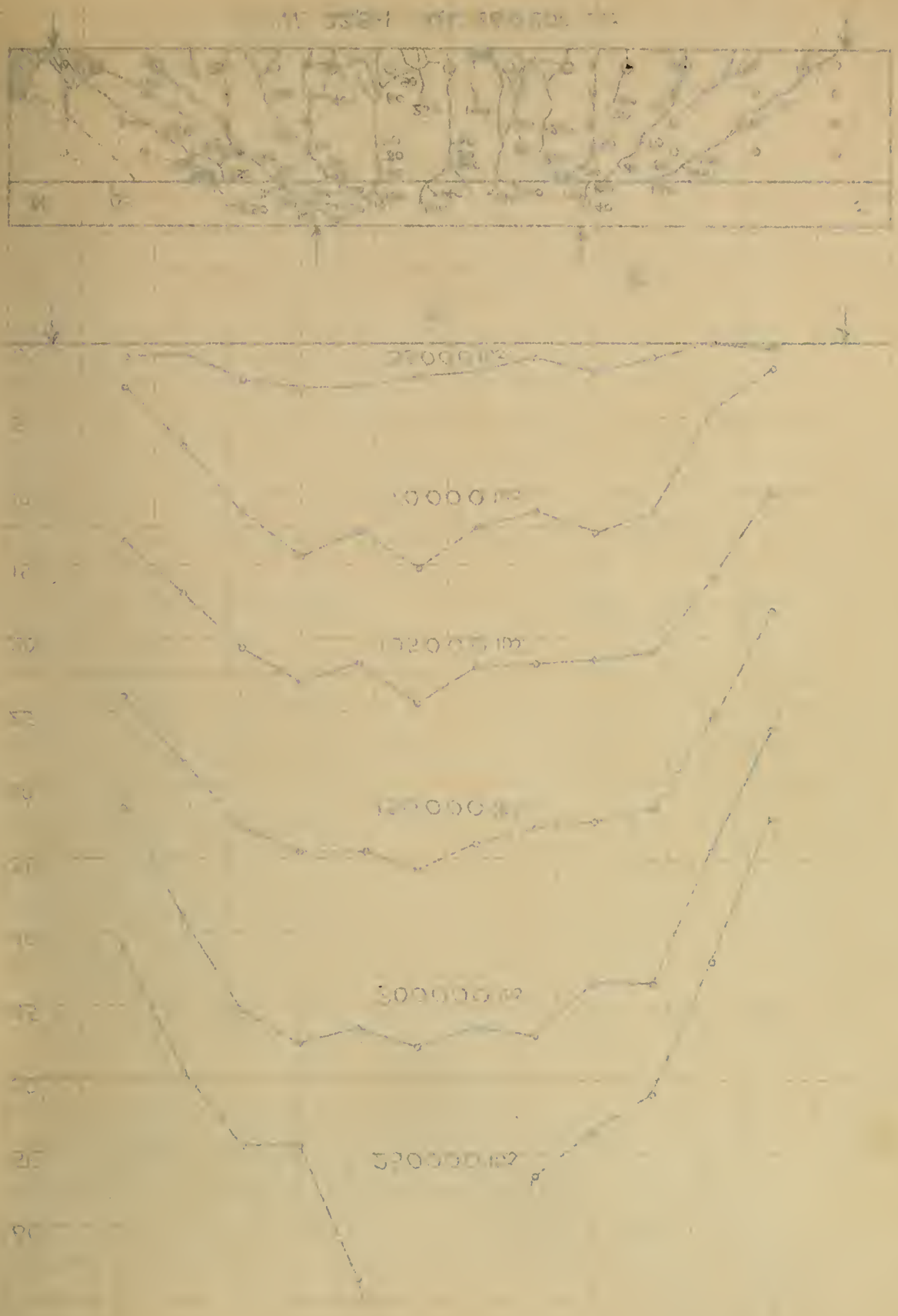

Unit Stress in Thousands of Pounds
30
20
10
0
240000 lbs.
20
10
0
200 000 lbs.
20
10
0
150 000 lbs.
10
0
105000 lbs.
10
0
70000 lbs.
5
0
35000 lbs.
S
N
E. 22.7-2 U.L. 268 600 lbs

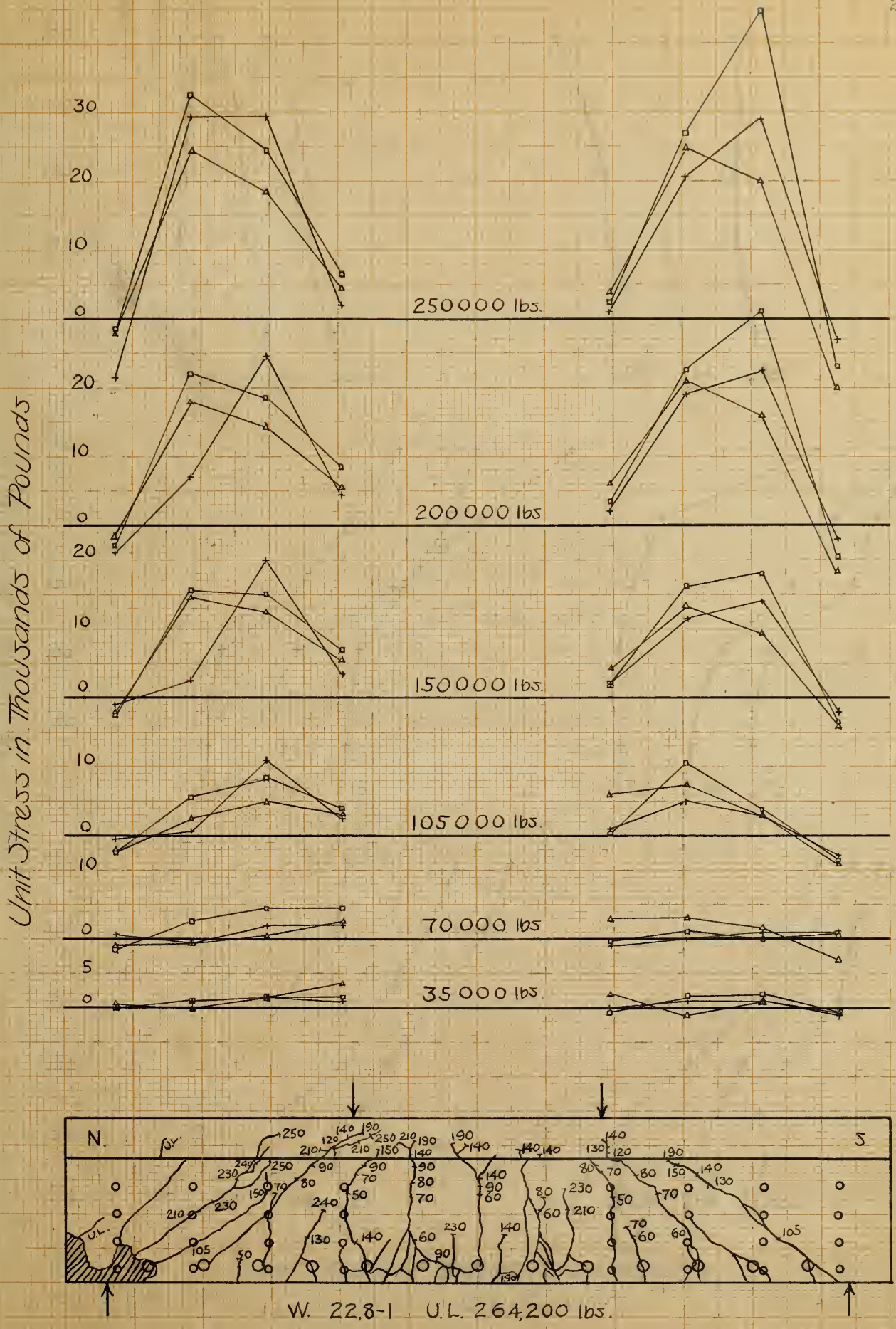

Load in Thousands of Pounds.
200
150
100
50
0
8
9
10
98
99
100
18
19
20
88
89
90
28
29
30
78
79
80
Unit Stress in Stirrups.
1 in. = 20000 lbs.

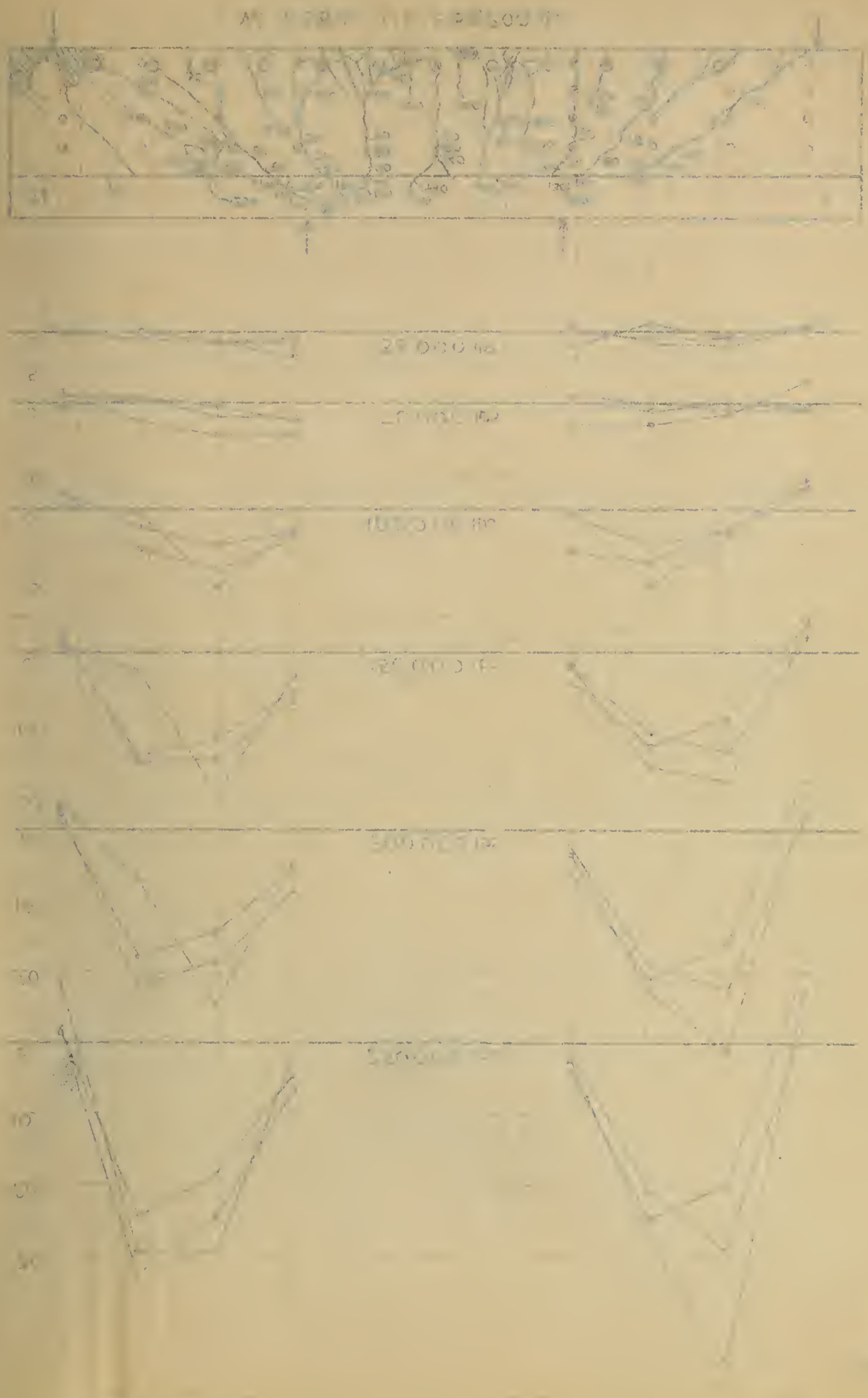


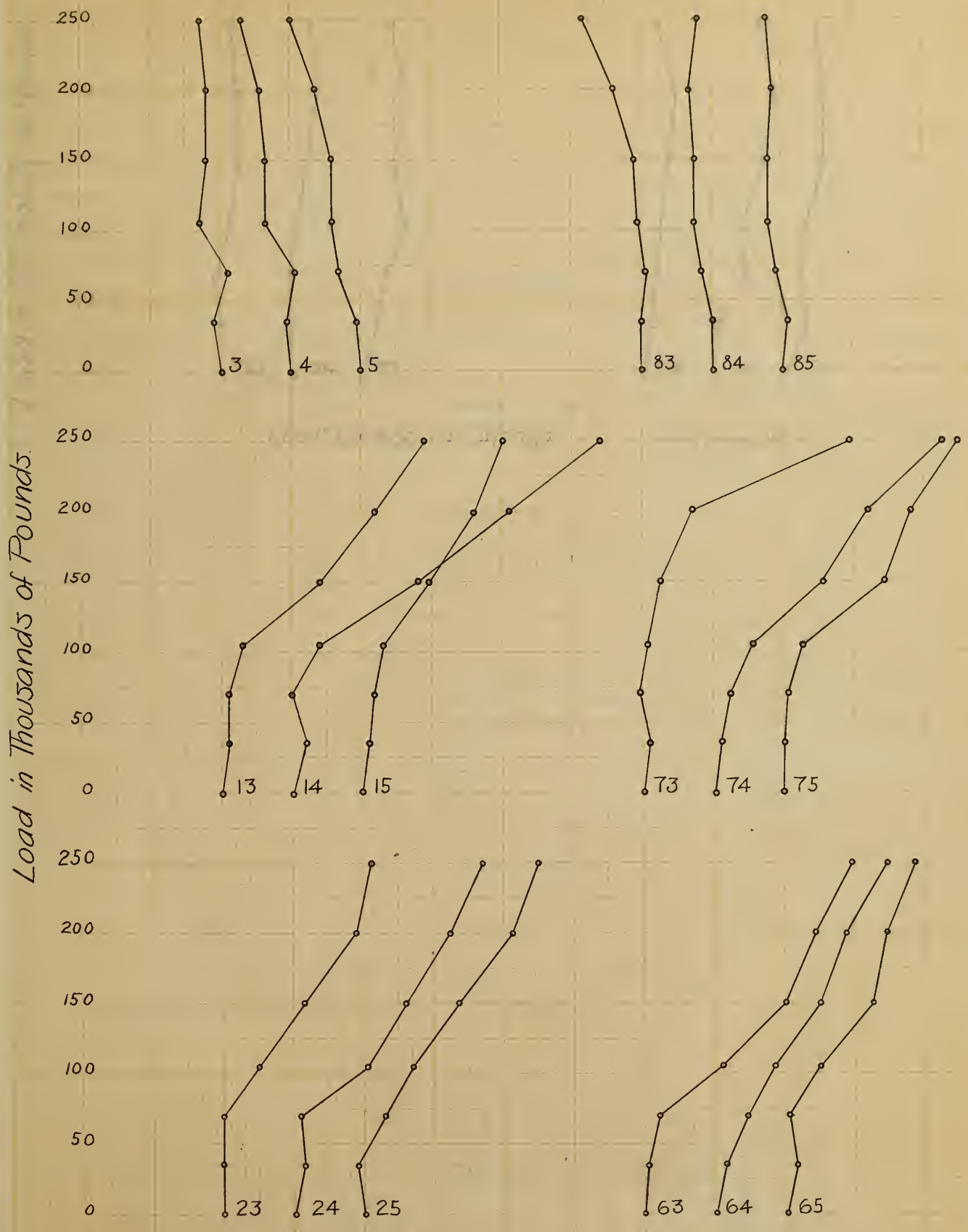
Unit Stress in Stirrups. 1 in = 20000 lbs.


Unit Stress in Thousands of Pounds
60
55
50
45
40
35
30
25
20
15
10
5
0
250000 lbs
200000 lbs
150000 lbs
105000 lbs.
70000 lbs.
35000 lbs.
N
S
U.L.
W. 22.8-1 U.L. 264200 Lbs.



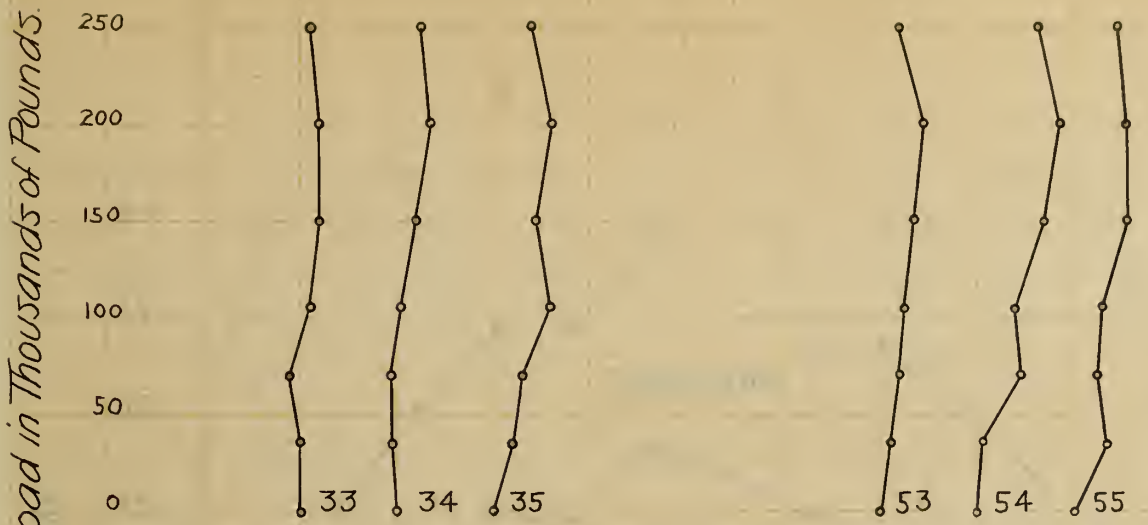

Unit Stress in Thousands of Pounds
250000 lbs.
200000 lbs
150000 lbs.
105000 lbs.
70000 lbs
35000 lbs.
N.
S.
U.L.
W. 22,8-1 U.L. 264,200 lbs.



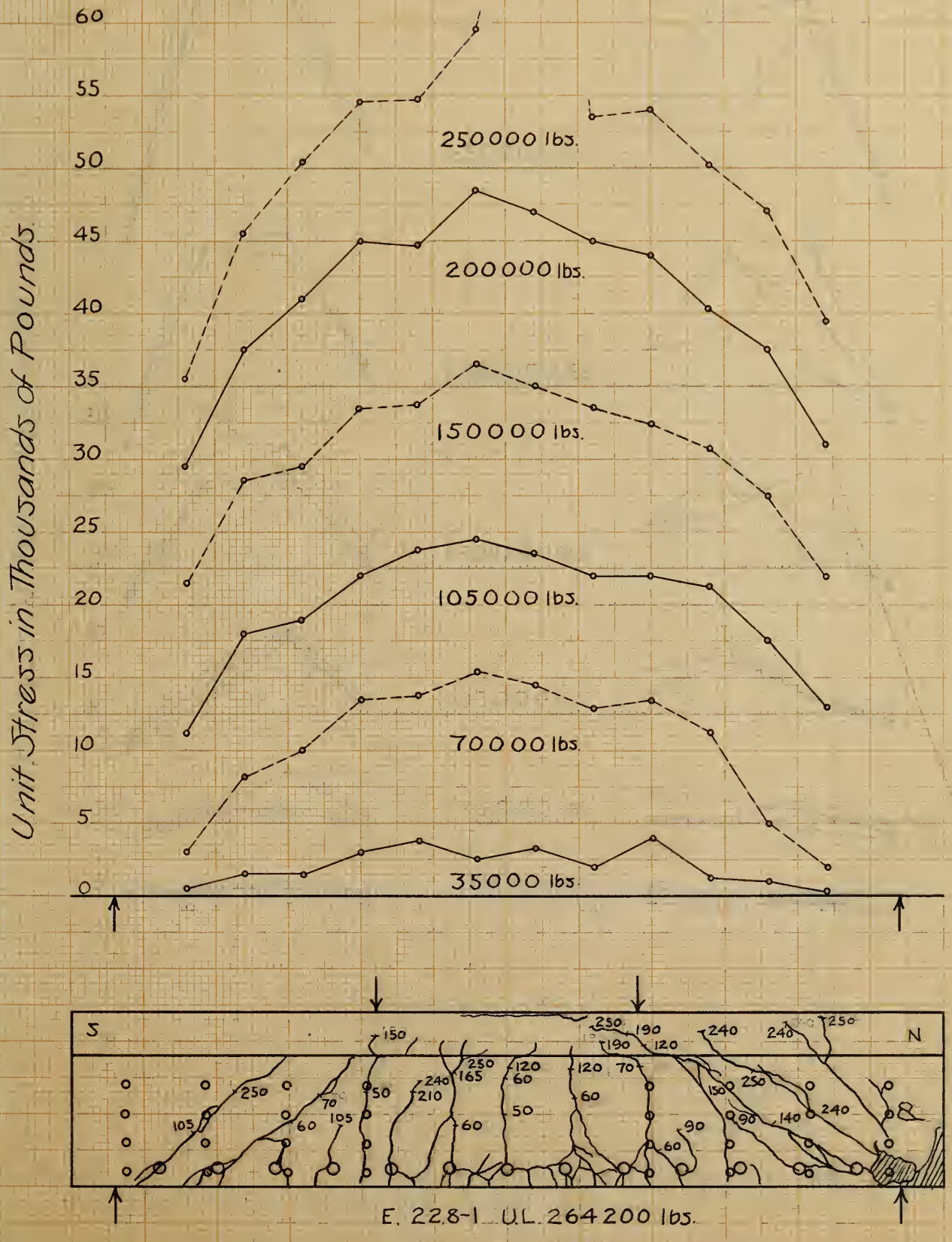


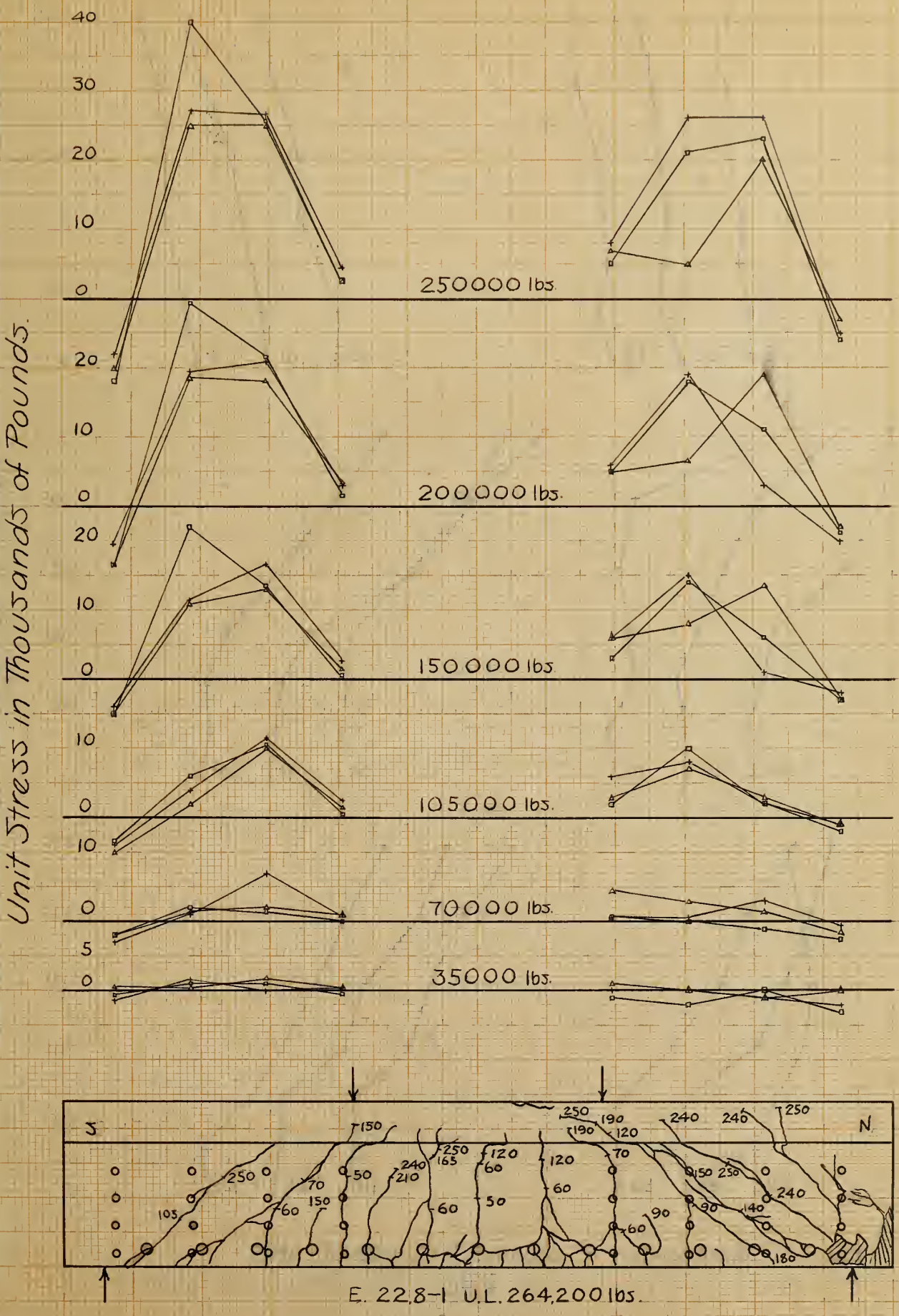
Unit Stress in Stirrups. 1 in. = 20000 lbs.

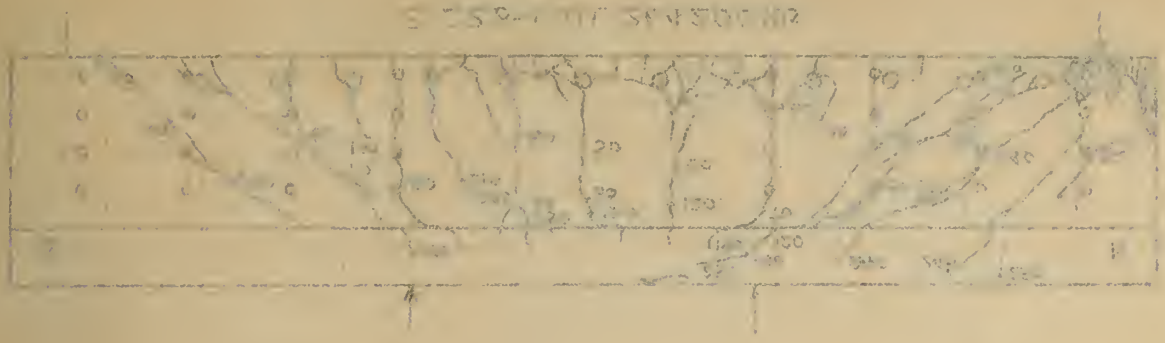
228-1 W.

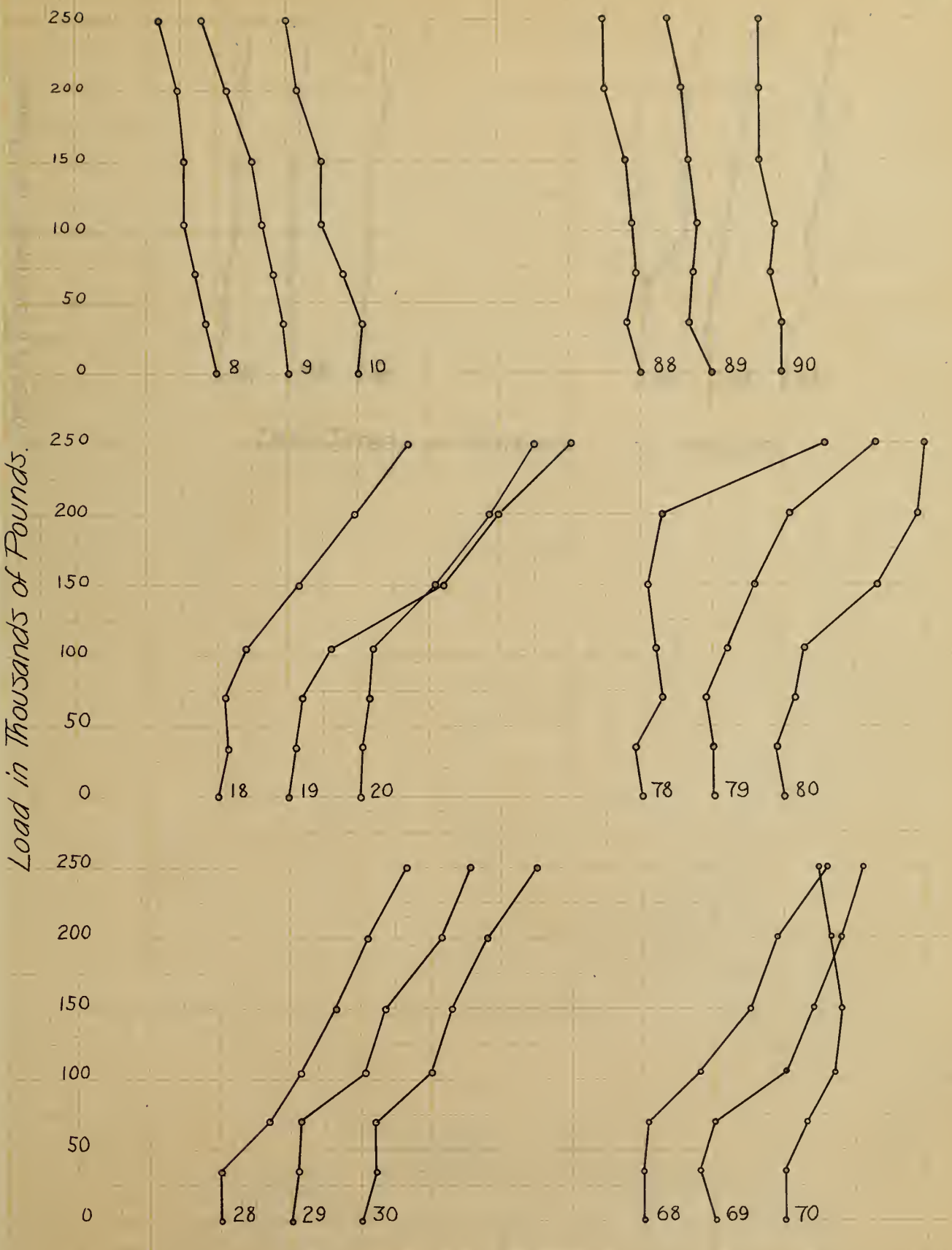


Unit Stress in Stirrups. 1 in = 20000 lbs.


Unit Stress in Thousands of Pounds
60
55
50
45
40
35
30
25
20
15
10
5
0
250000 lbs.
200000 lbs.
150000 lbs.
105000 lbs.
70000 lbs.
35000 lbs.
S
N
E. 22.8-1 U.L. 264200 lbs.

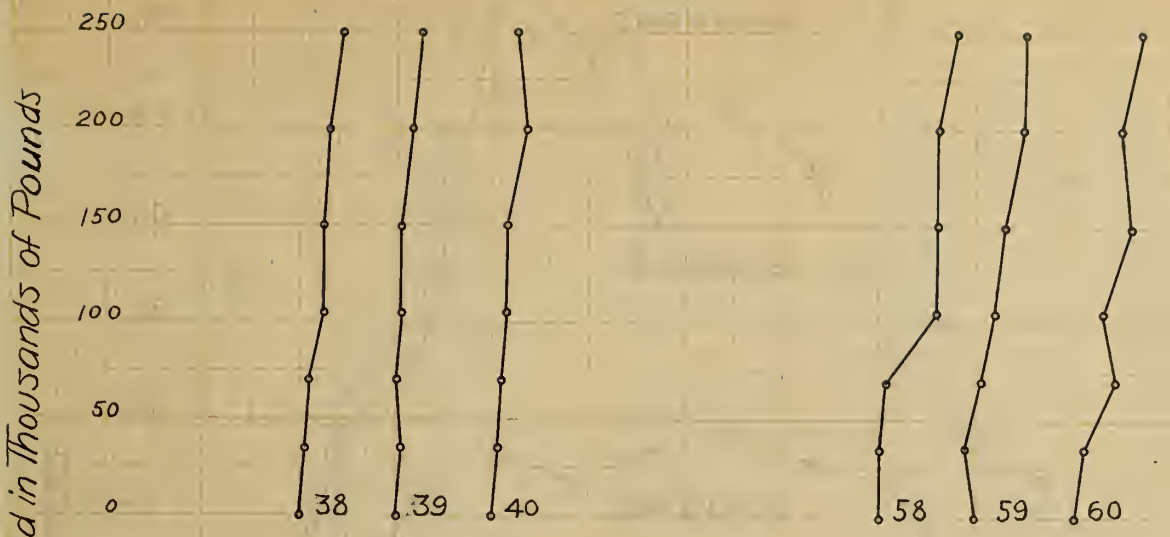

Unit Stress in Thousands of Pounds.
40
30
20
10
0
250000 lbs.
20
10
0
200000 lbs.
20
10
0
150000 lbs.
10
0
105000 lbs.
10
0
70000 lbs.
5
0
35000 lbs.
S
N
E. 22.8-1 U.L. 264,200 lbs.



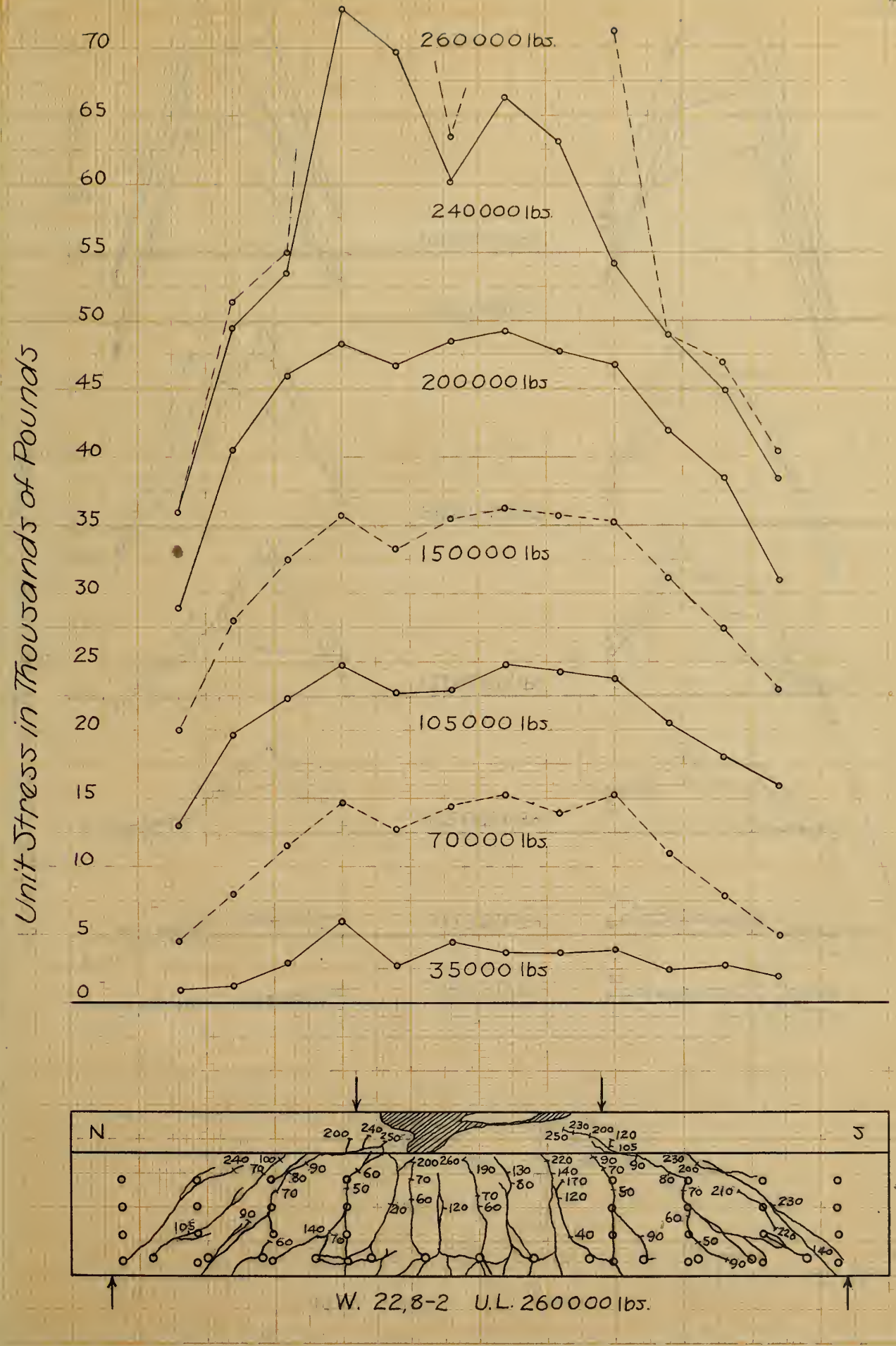


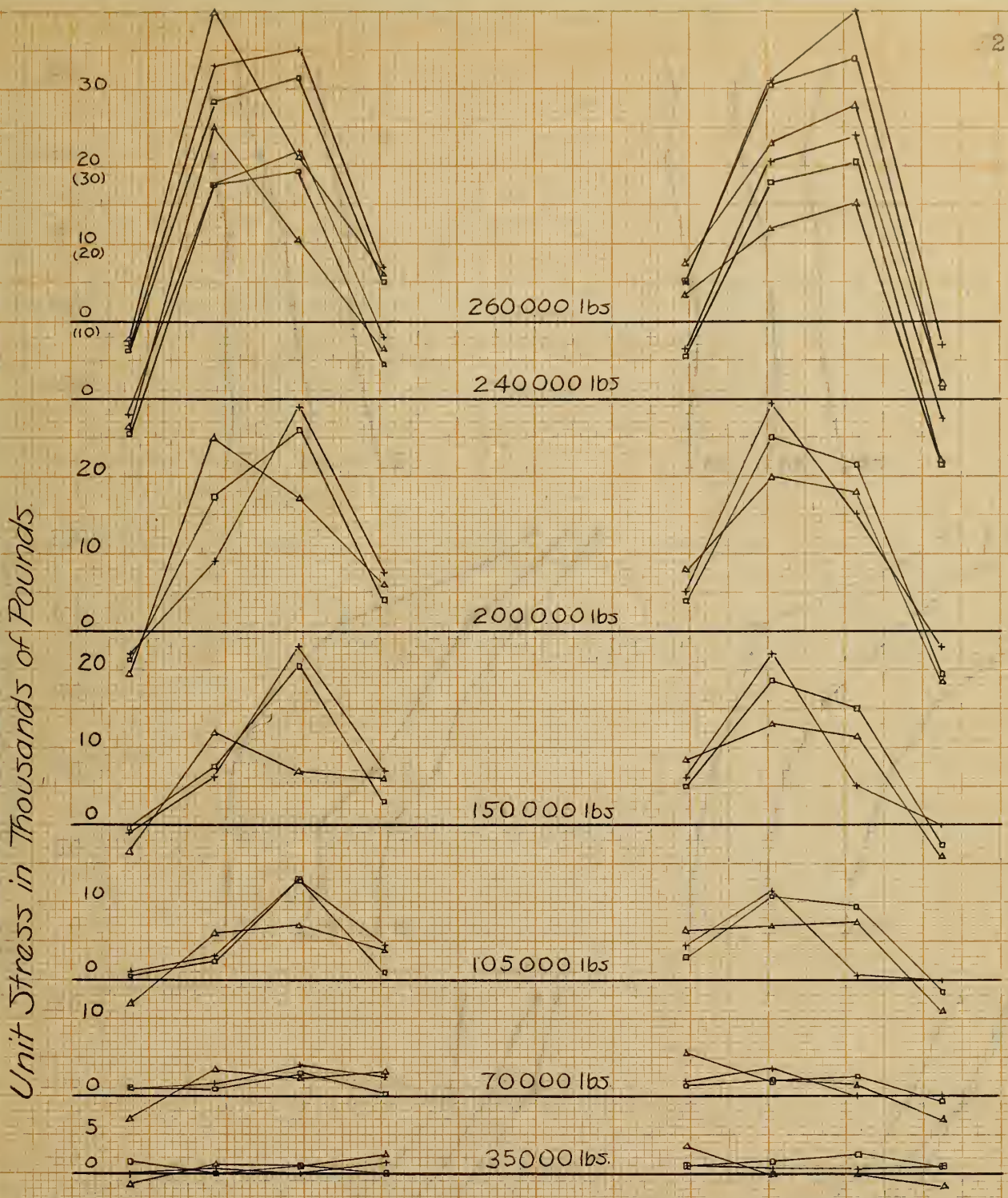
Unit Stress in Stirrups. 1 in. = 20000 lbs.

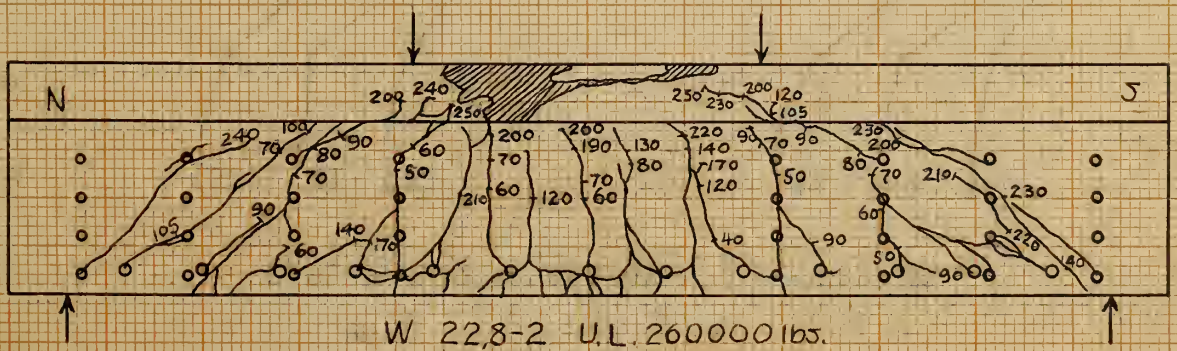
22,8-1 E.



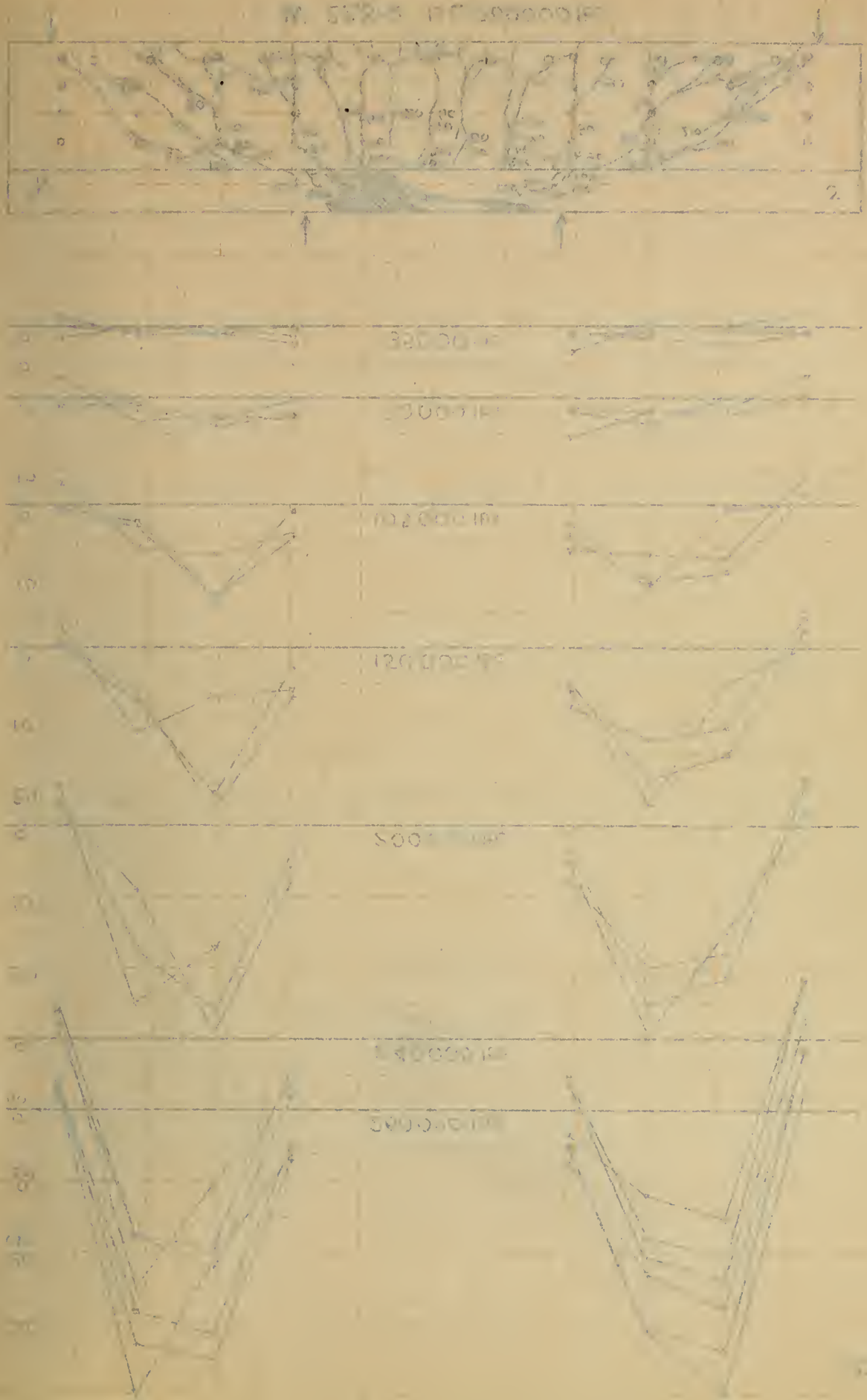
Unit Stress in Stirrups. 1 in = 20000 lbs.

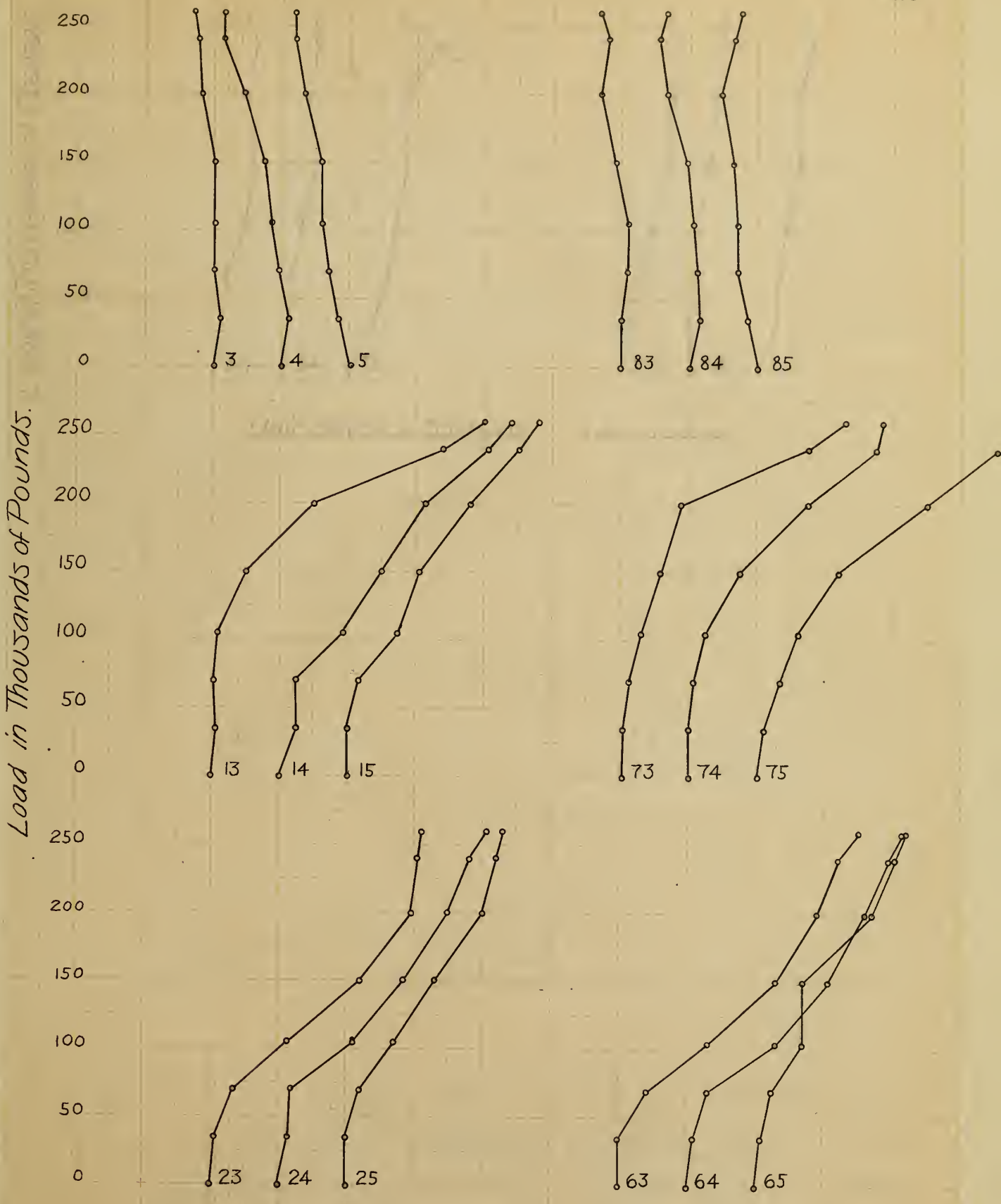

Unit Stress in Thousands of Pounds
70
65
60
55
50
45
40
35
30
25
20
15
10
5
0
260000 lbs.
240000 lbs.
200000 lbs
150000 lbs
105000 lbs.
70000 lbs.
35000 lbs
N
S
W. 22,8-2 U.L. 260000 lbs.





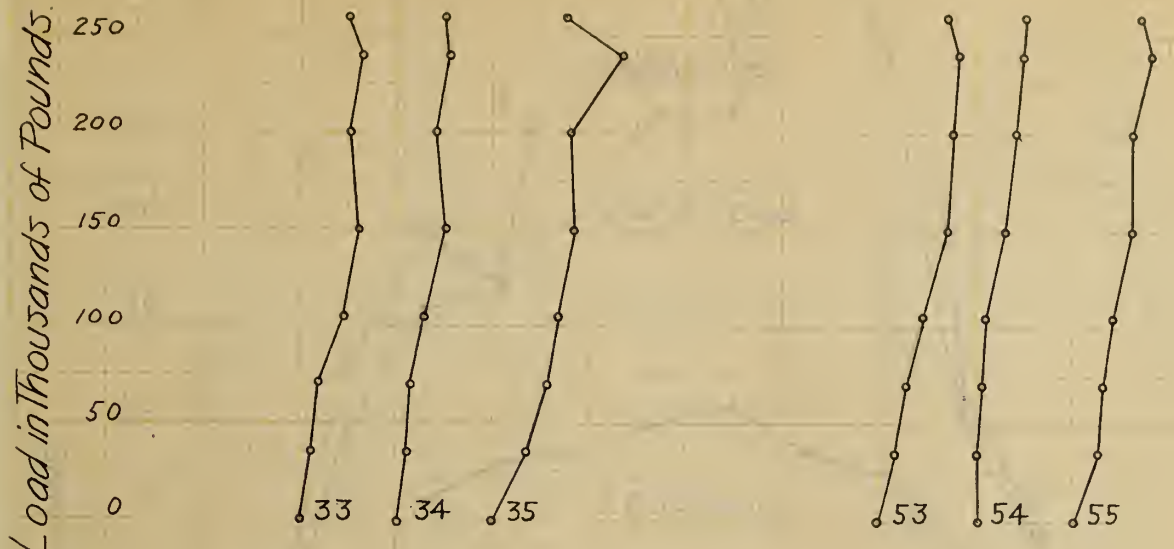
W 22,8-2 U.L. 260000 lbs.



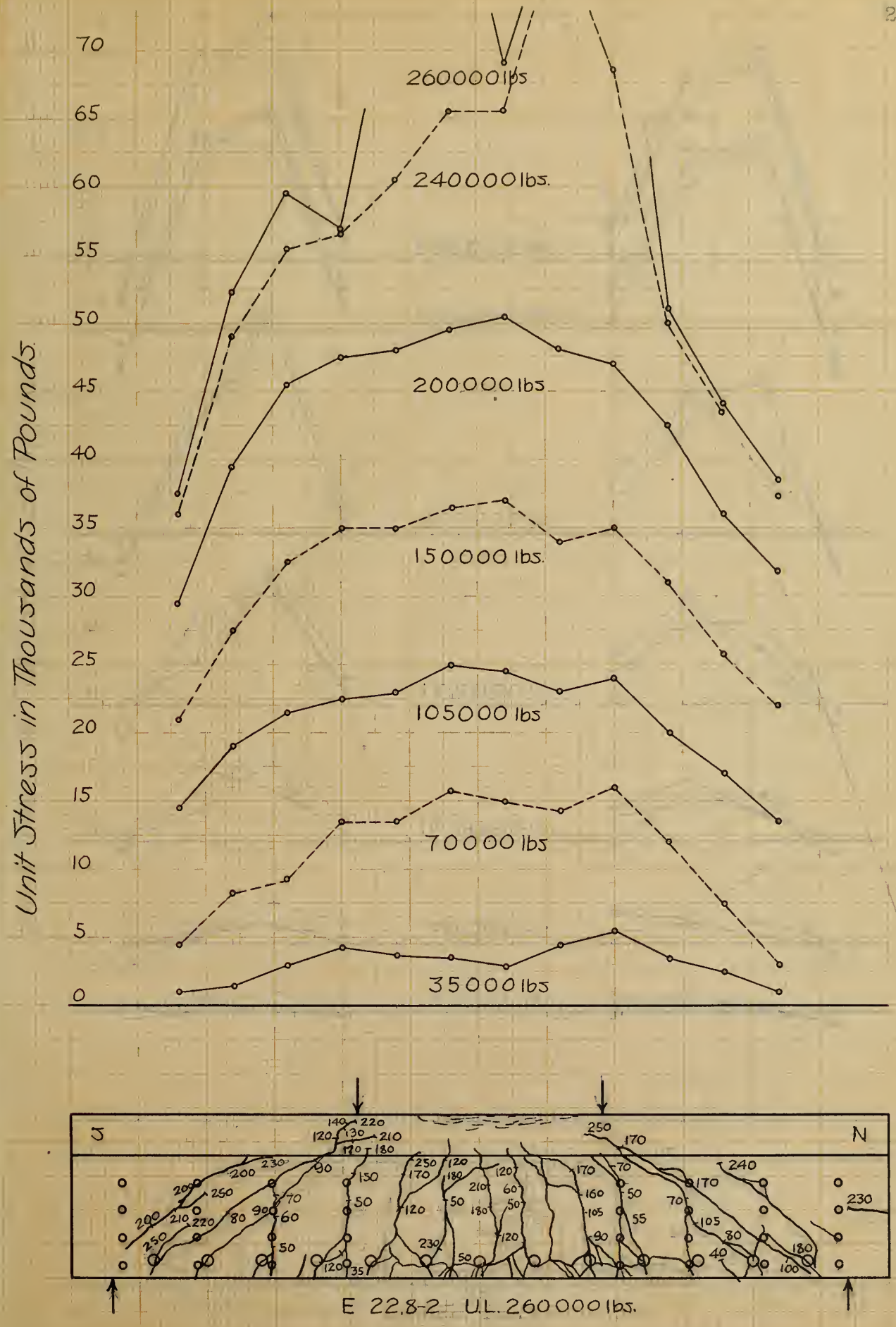


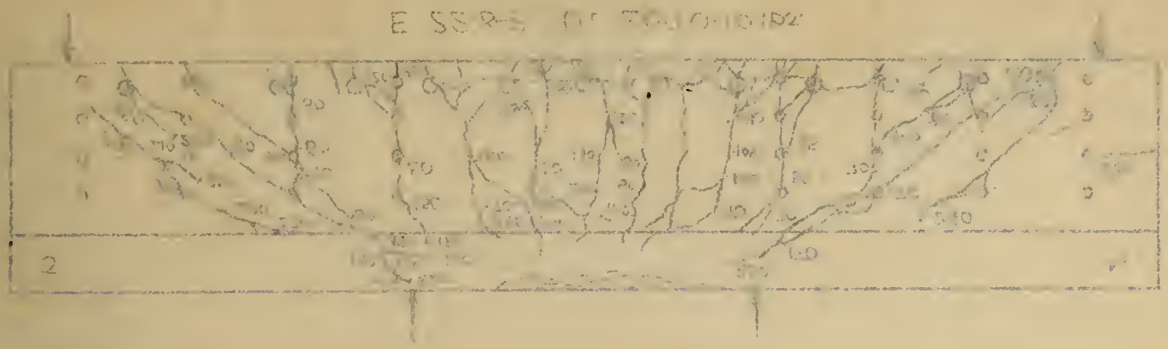
Unit Stress in Stirrups. 1 in = 20000 lbs.

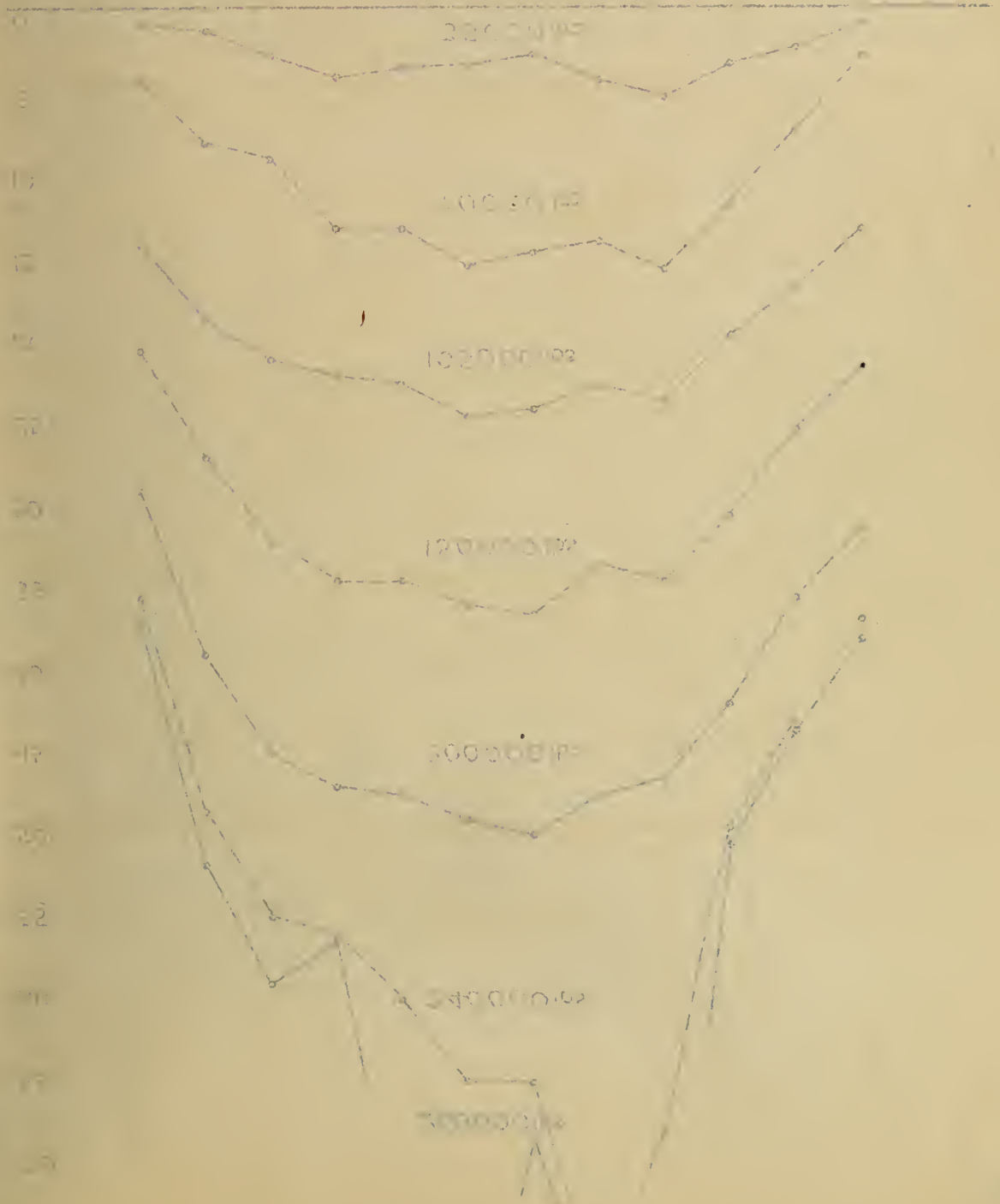
22,8-2 W.

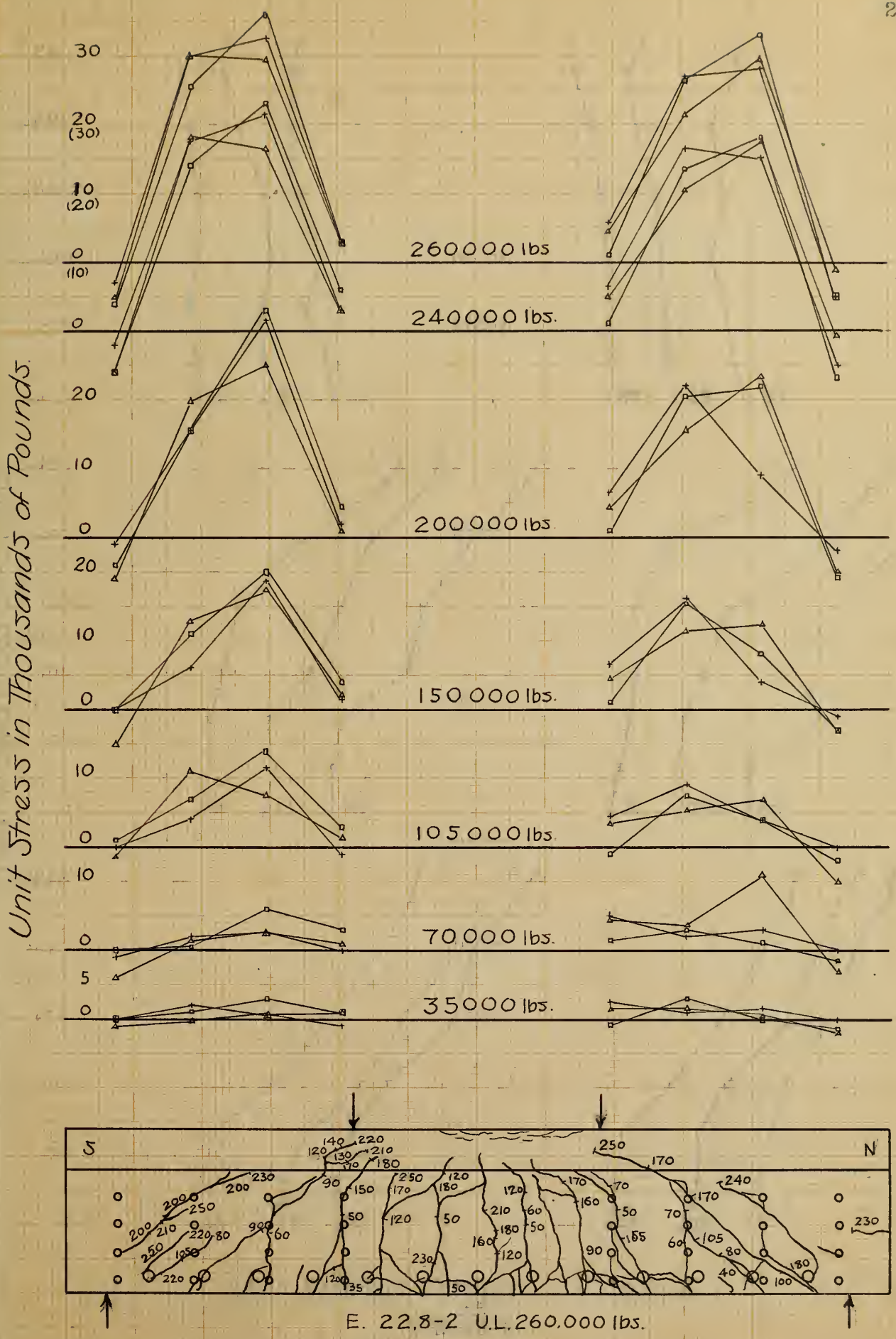


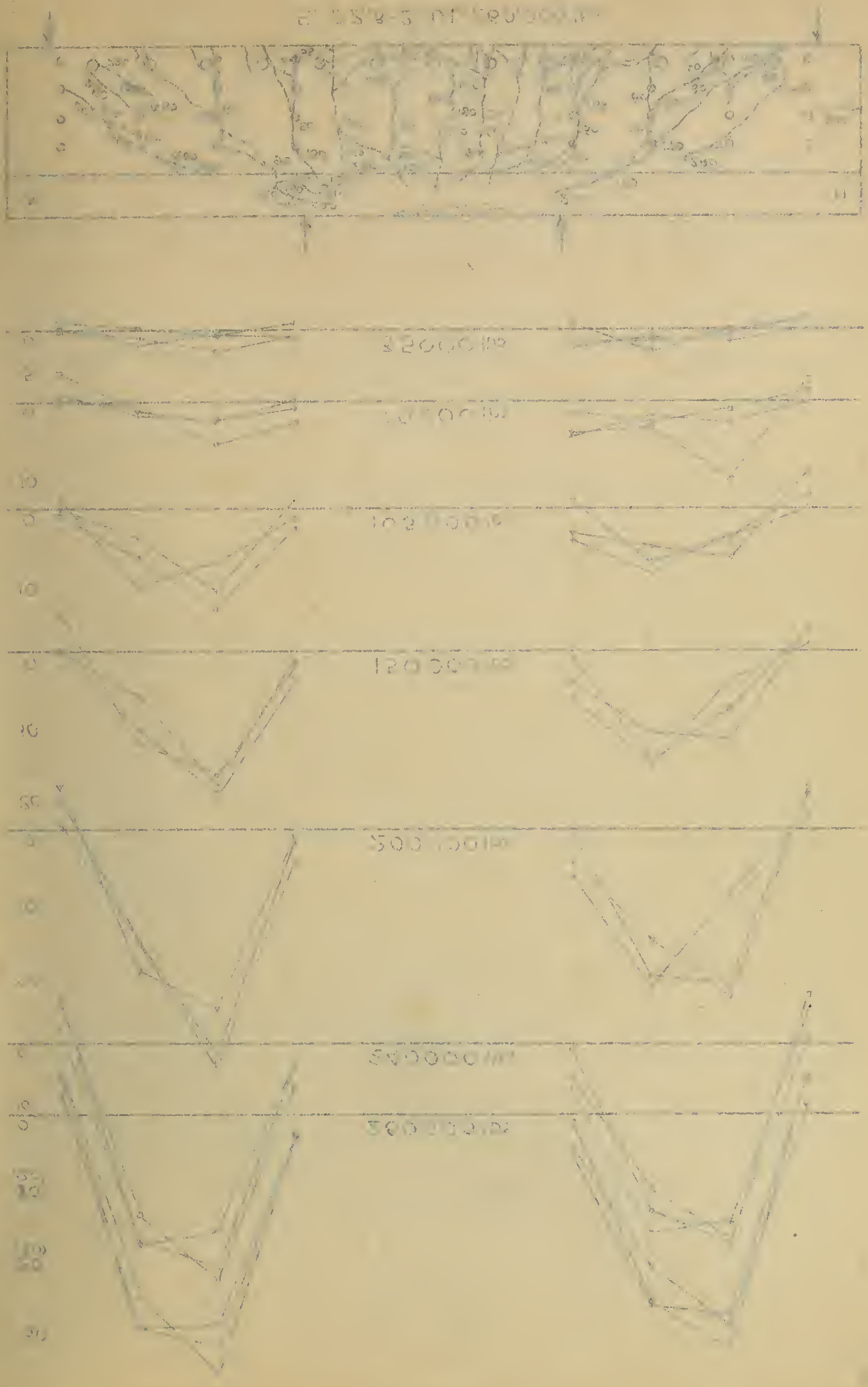
Unit Stress in Stirrups. 1 in. = 20000 lbs.

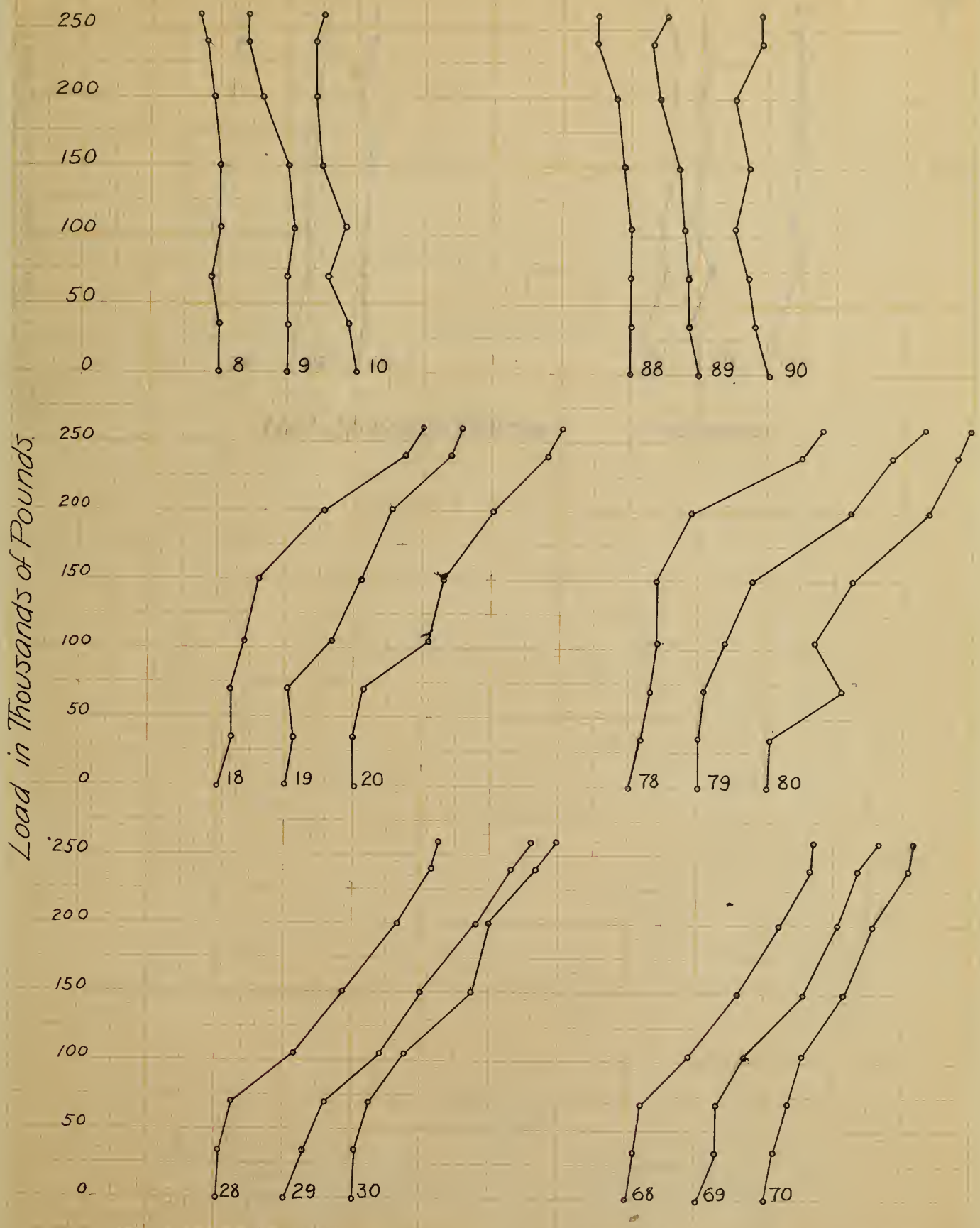

Unit Stress in Thousands of Pounds
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
260000 lbs
240000 lbs.
200000 lbs
150000 lbs.
105000 lbs
70000 lbs
35000 lbs
S
N
E 22.8-2 U.L. 260000 lbs.





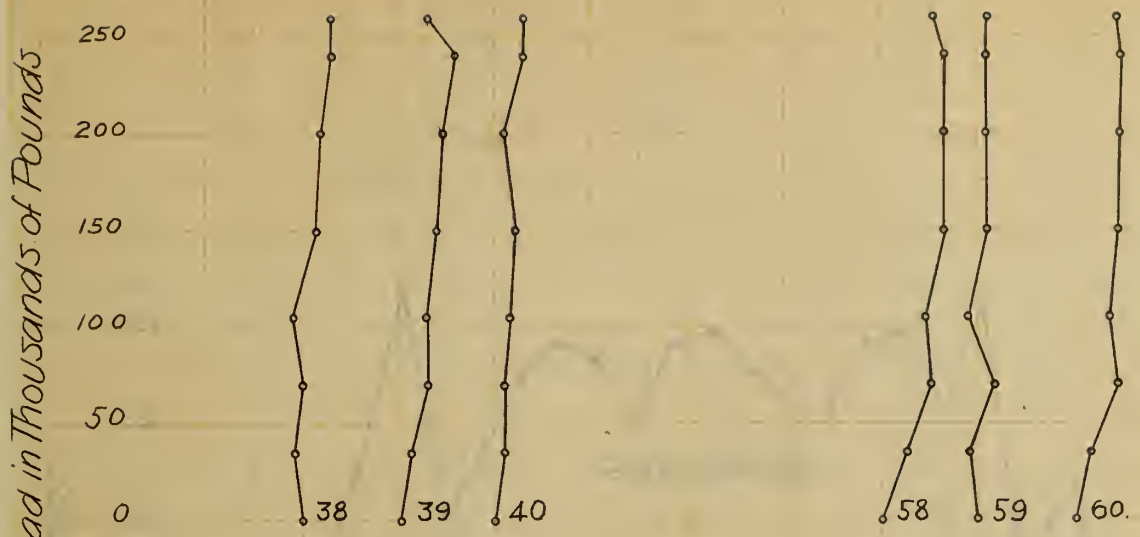

Unit Stress in Thousands of Pounds
260000 lbs
240000 lbs.
200000 lbs.
150000 lbs.
105000 lbs.
70000 lbs.
35000 lbs.
S
N
E. 22.8-2 U.L. 260,000 lbs.





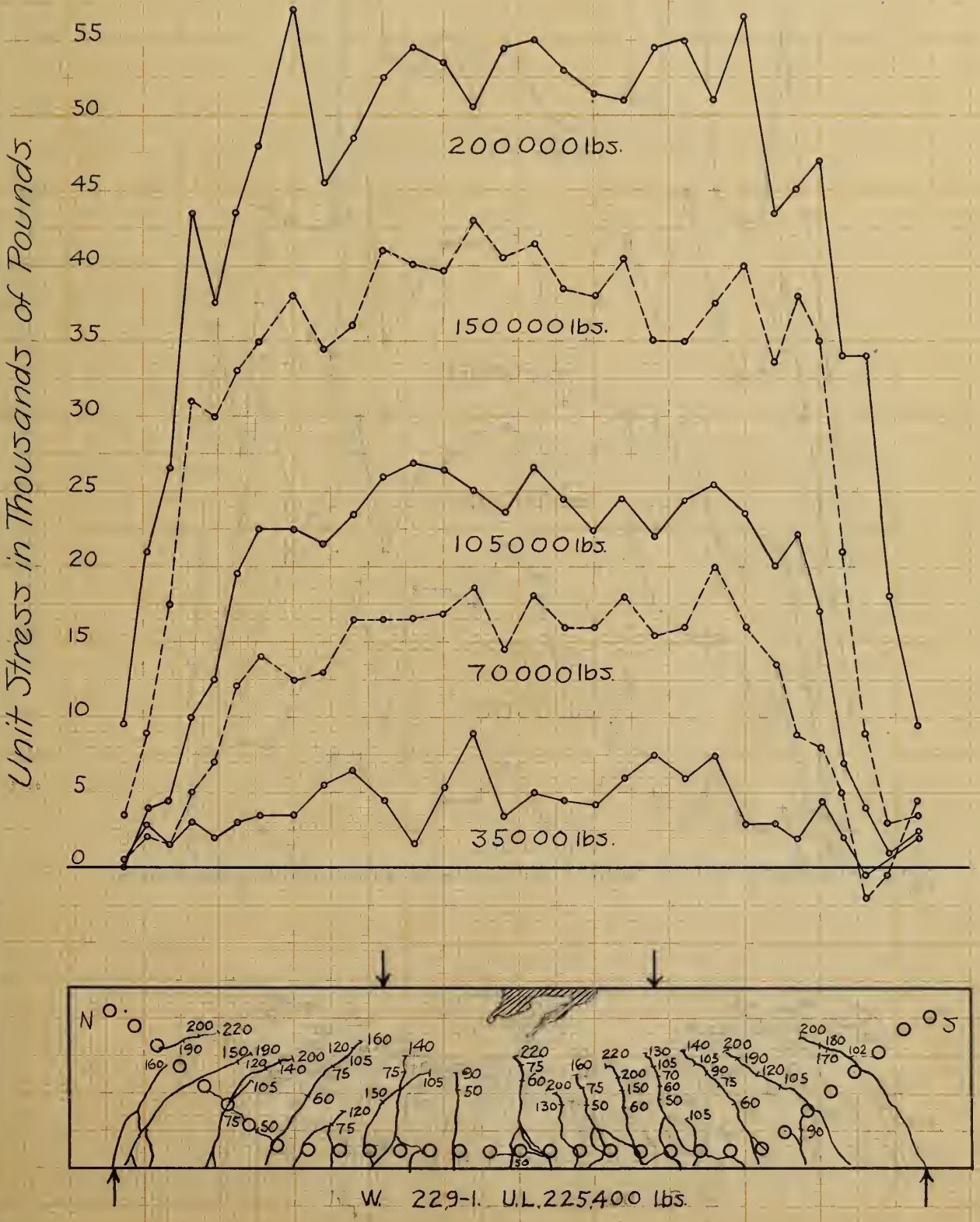
Unit Stress in Stirrups. 1 in. = 20000 lbs

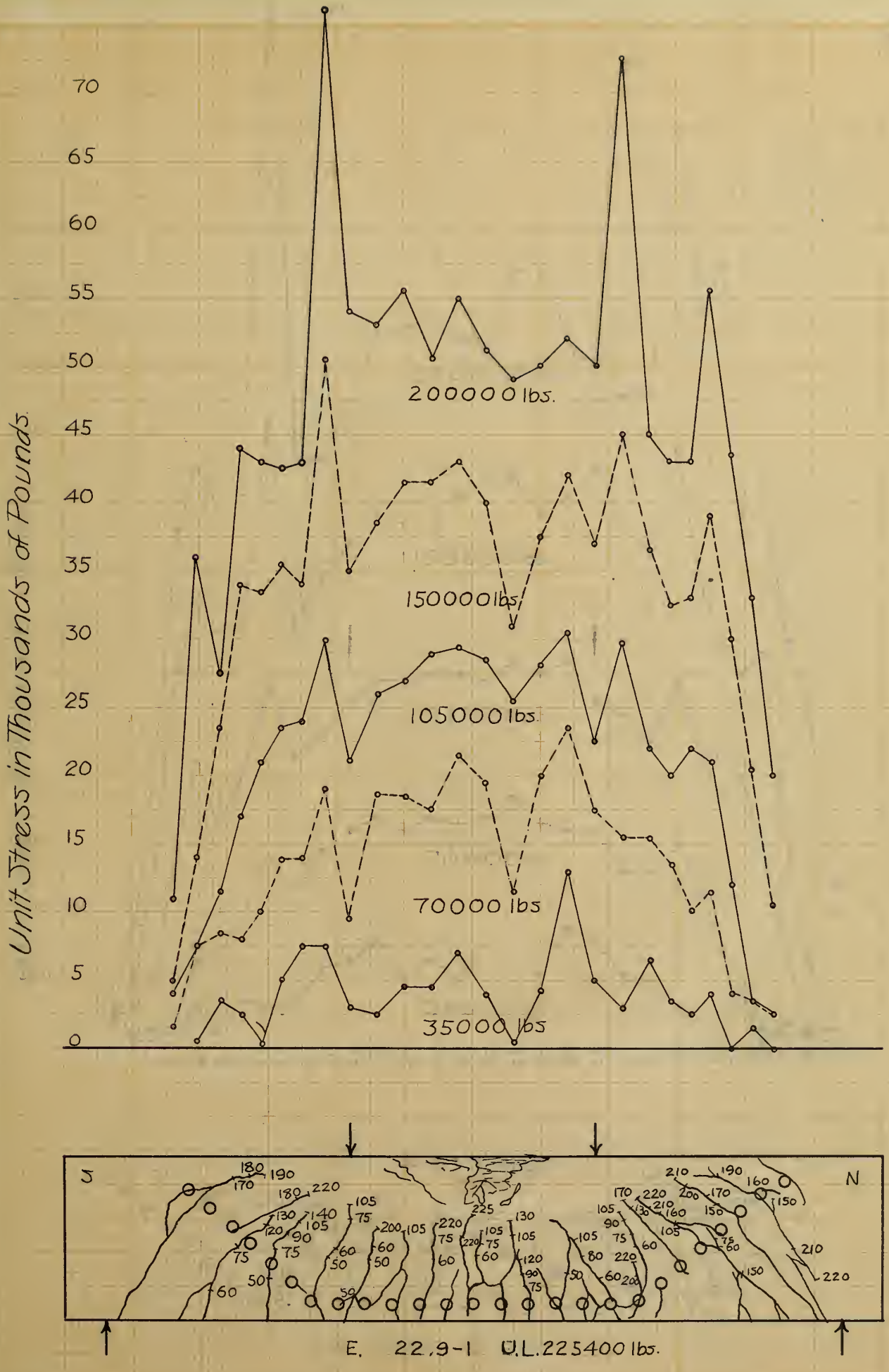
22.8-2 E.

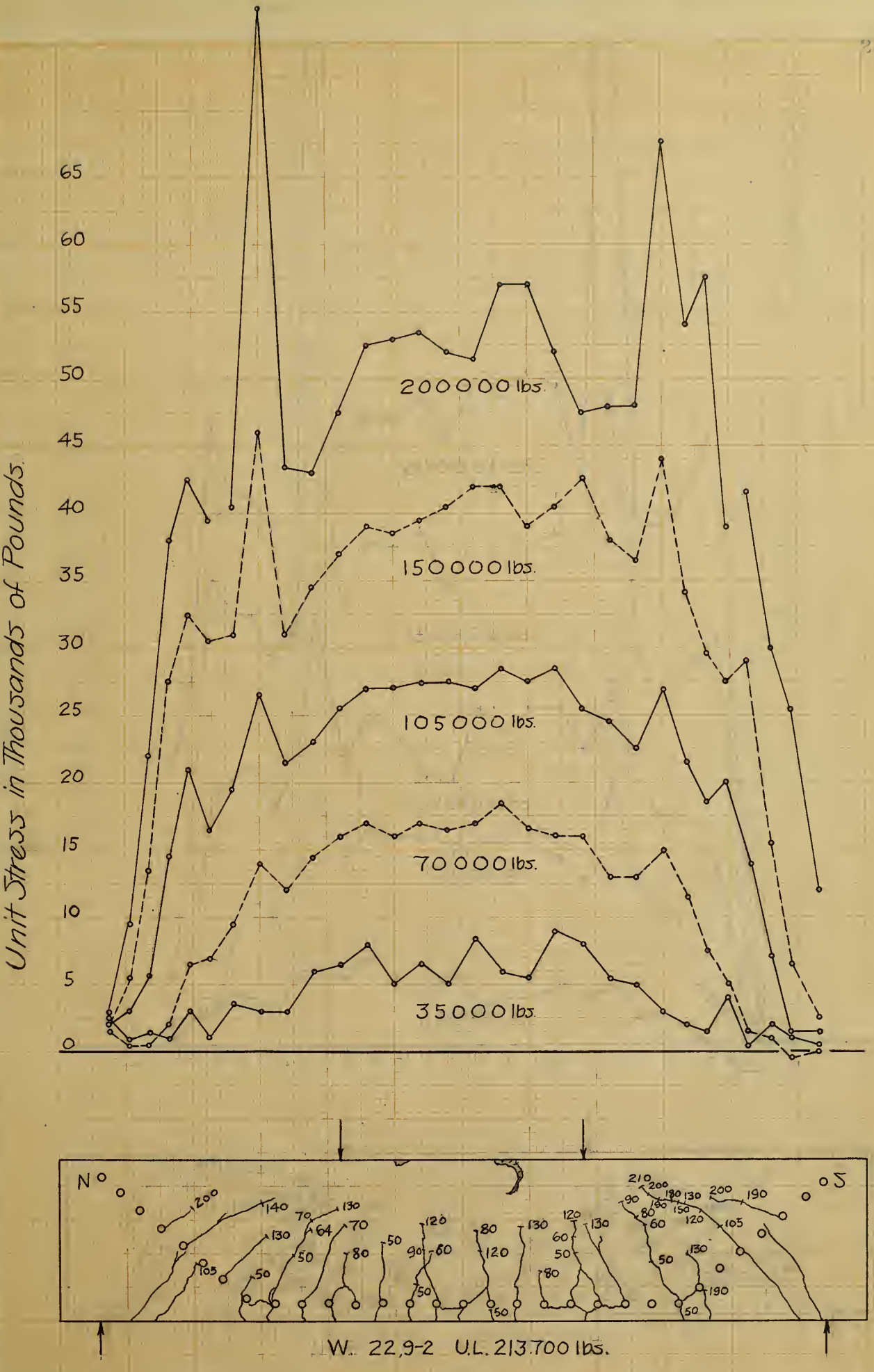


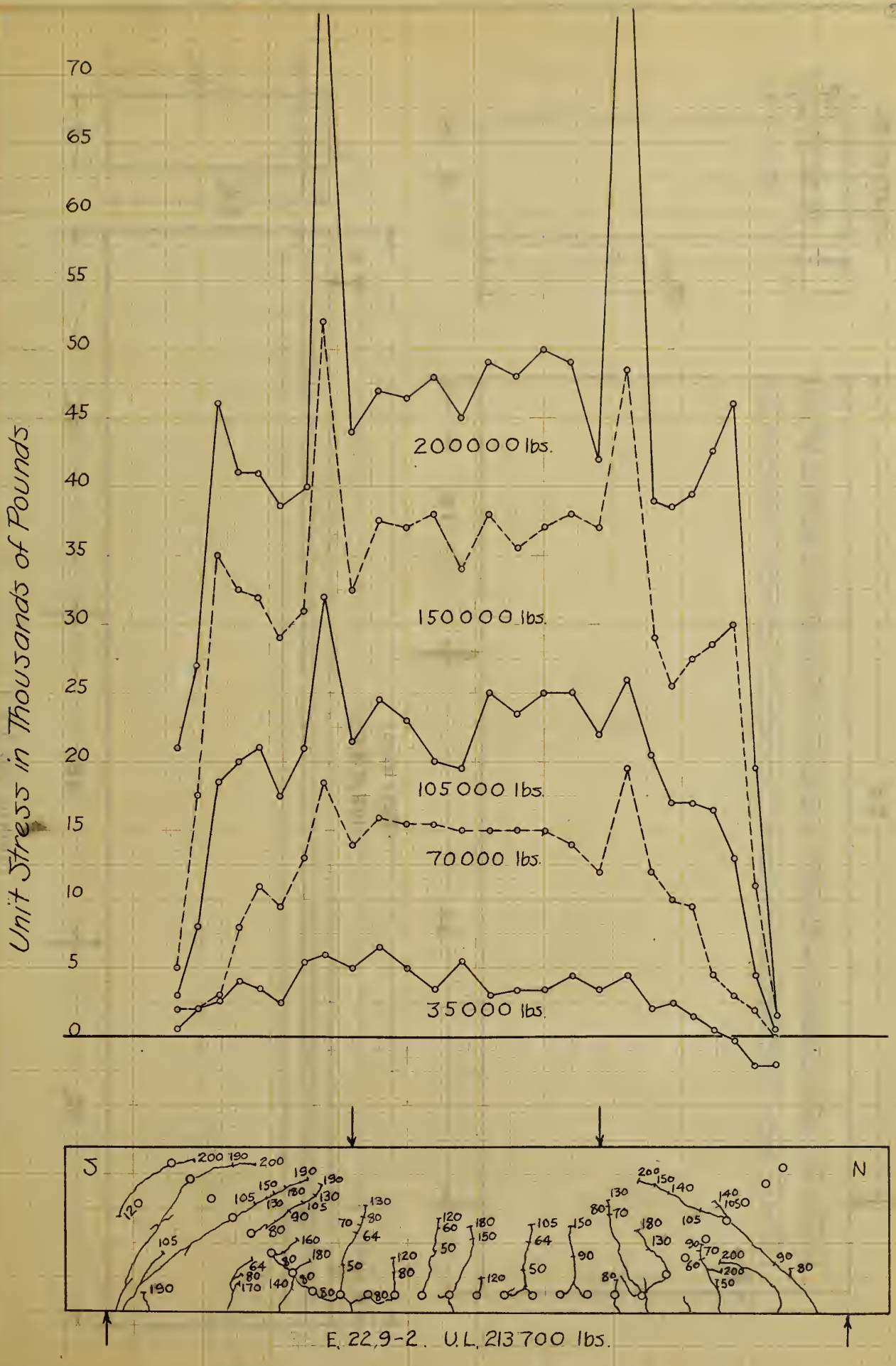
Unit Stress in Stirrups 1 in. = 20000 lbs.

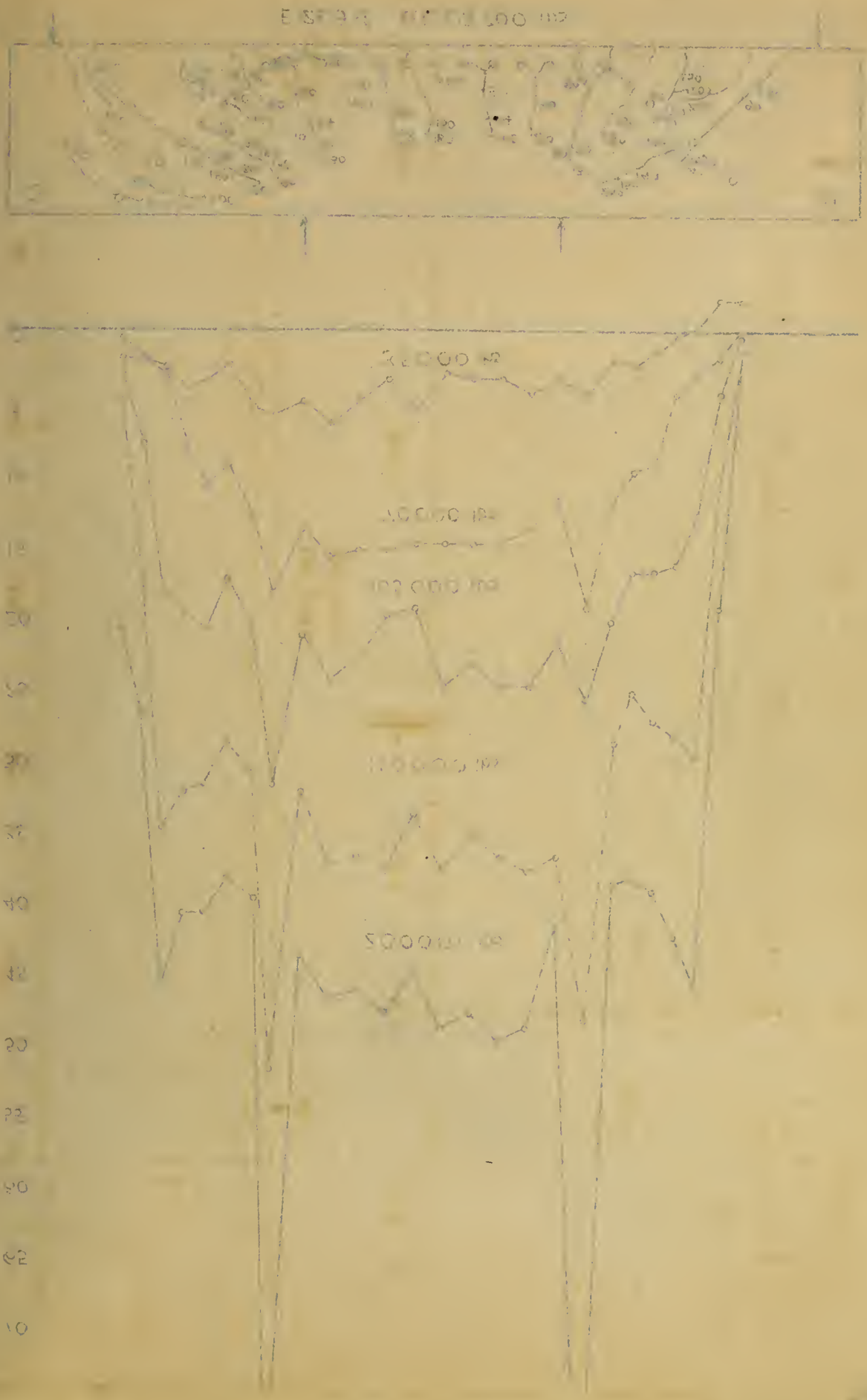
22,8-2 E.


Unit Stress in Thousands of Pounds
55
50
45
40
35
30
25
20
15
10
5
0
200000 lbs.
150000 lbs.
105000 lbs.
70000 lbs.
35000 lbs.
N
S
W. 229-1. U.L. 225,400 lbs.


Unit Stress in Thousands of Pounds
70
65
60
55
50
45
40
35
30
25
20
15
10
5
0
200000 lbs.
150000 lbs.
105000 lbs.
70000 lbs
35000 lbs
S
N
E. 22.9-1 U.L. 225400 lbs.


Unit Stress in Thousands of Pounds.
65
60
55
50
45
40
35
30
25
20
15
10
5
0
200000 lbs.
150000 lbs.
105000 lbs.
70000 lbs.
35000 lbs.
N
S
W. 22,9-2 U.L. 213.700 lbs.


Unit Stress in Thousands of Pounds
70
65
60
55
50
45
40
35
30
25
20
15
10
5
0
200000 lbs.
150000 lbs.
105000 lbs.
70000 lbs.
35000 lbs.
S
N
E. 22,9-2. U.L. 213700 lbs.



Beam No. 22.1

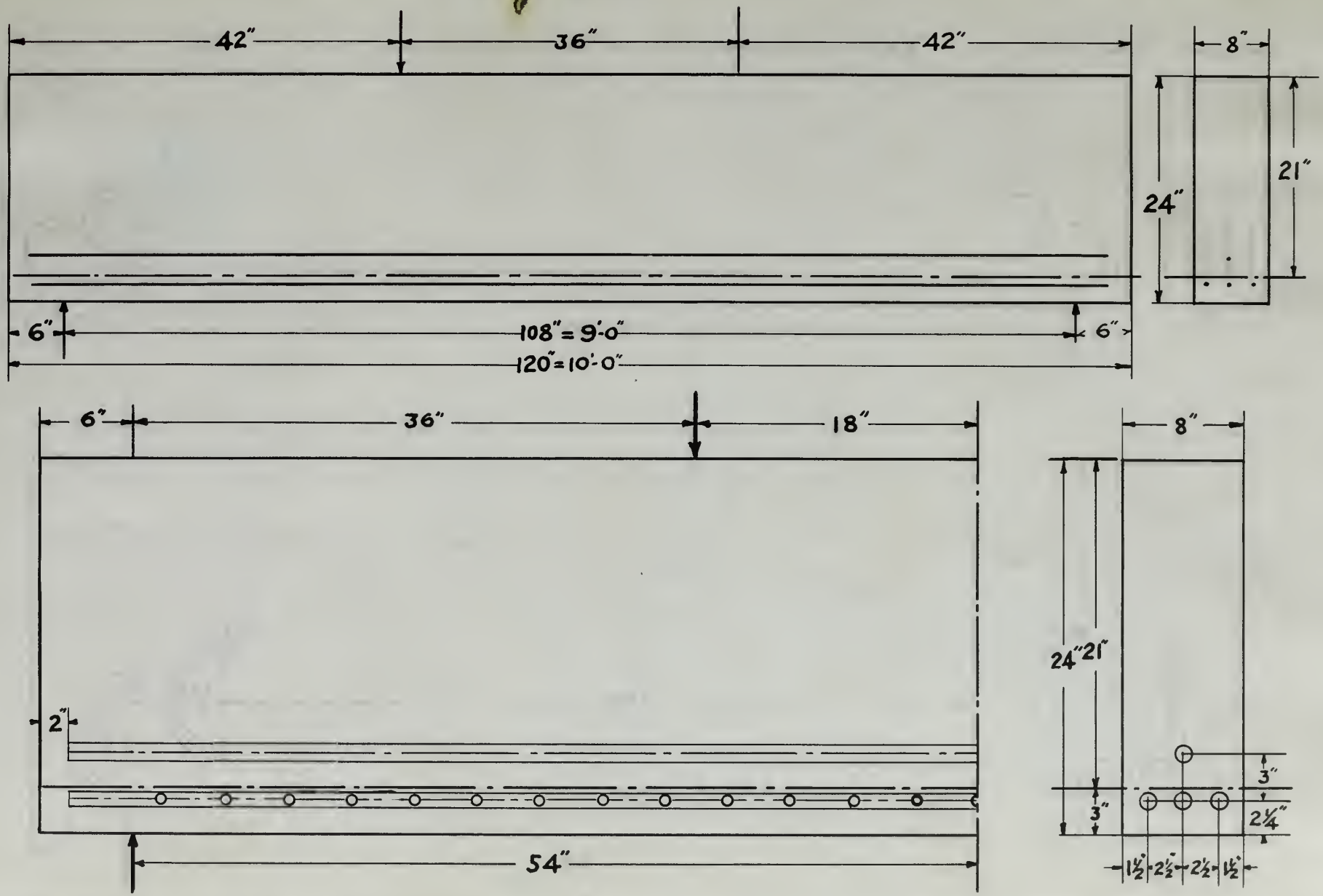
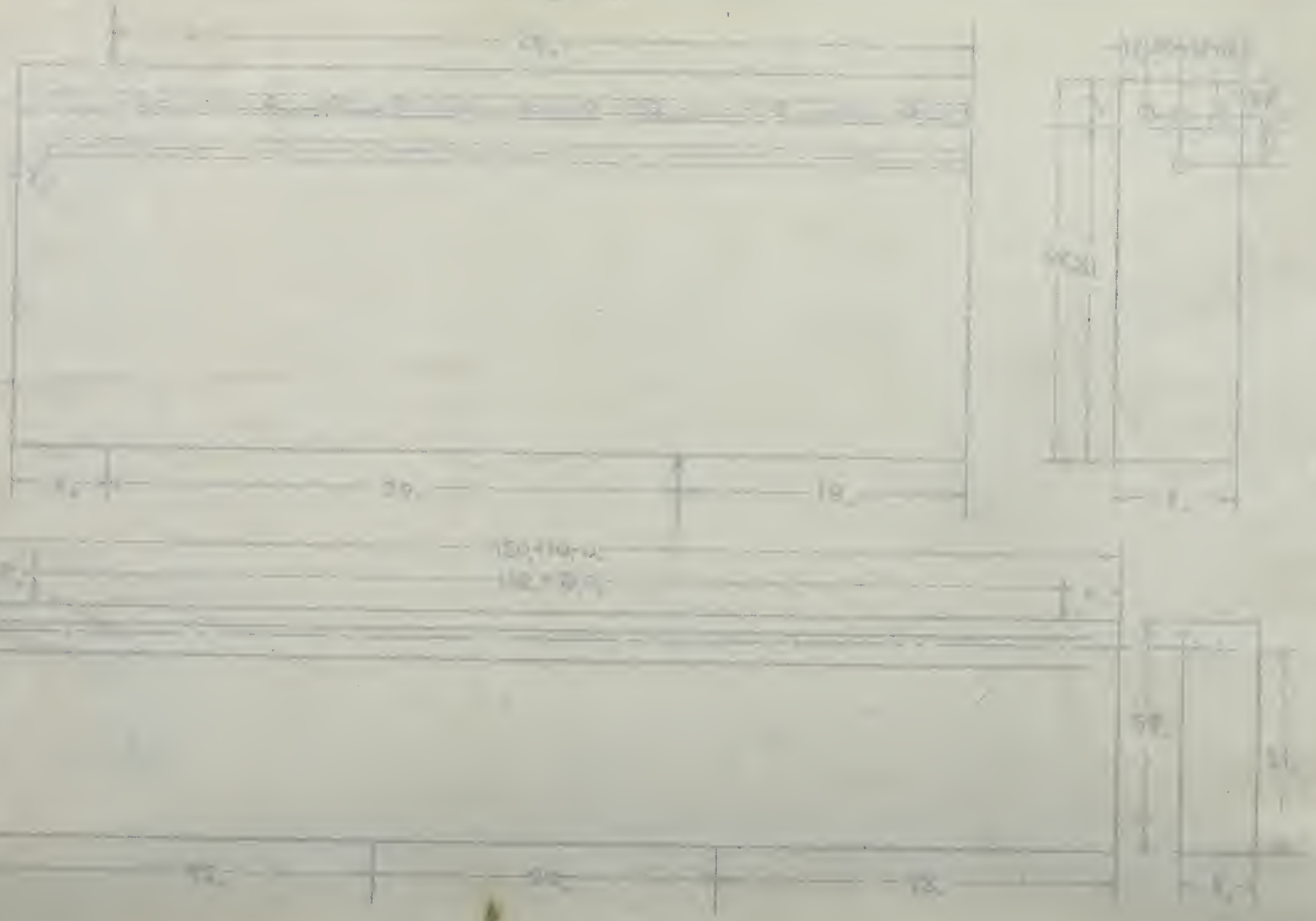


Fig. 11



Beam No. 22.2

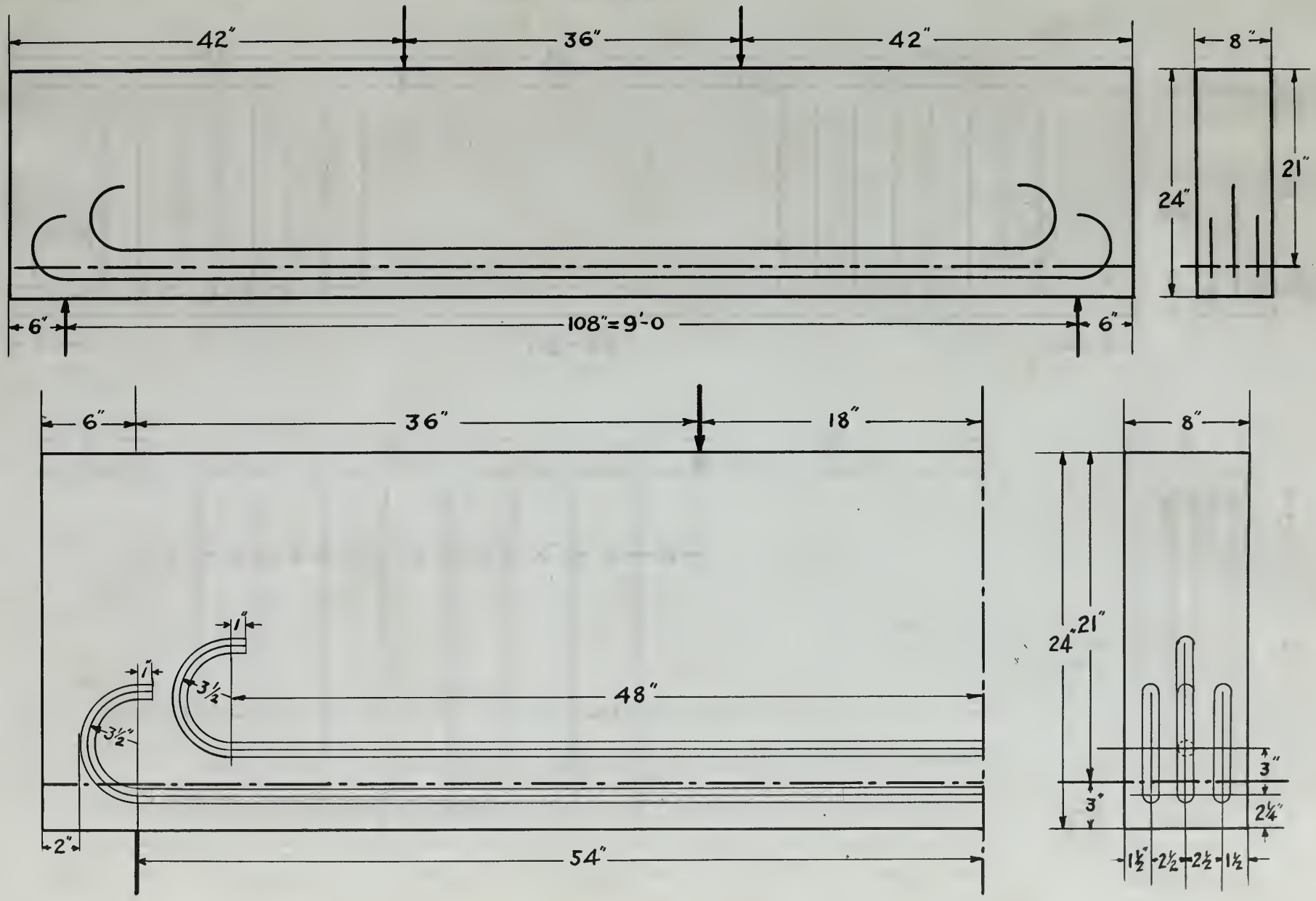
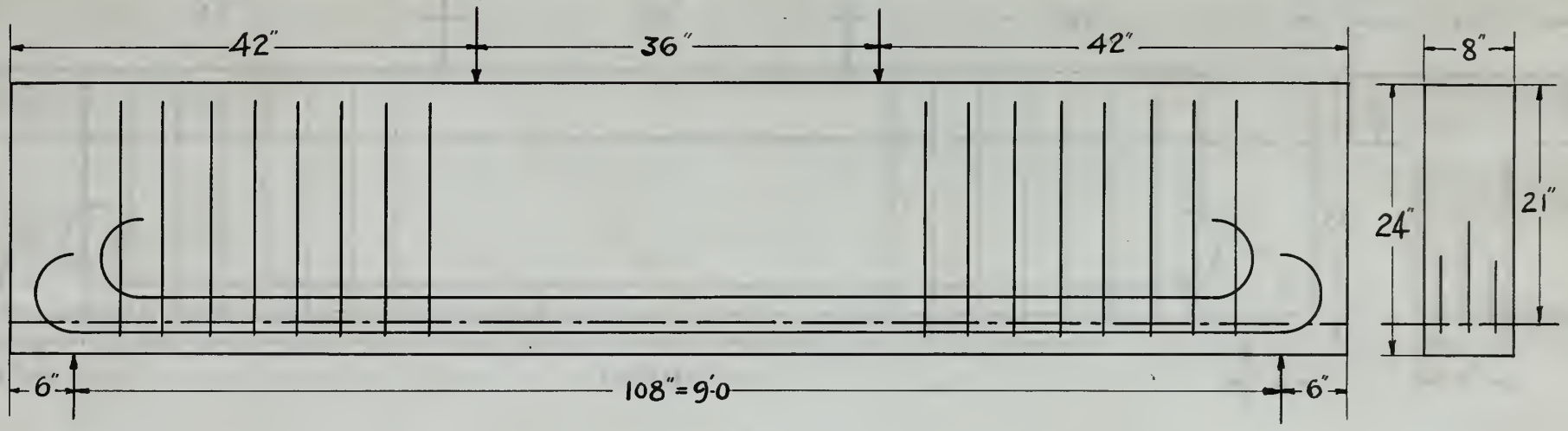


Fig 12

Beam No. 22,3



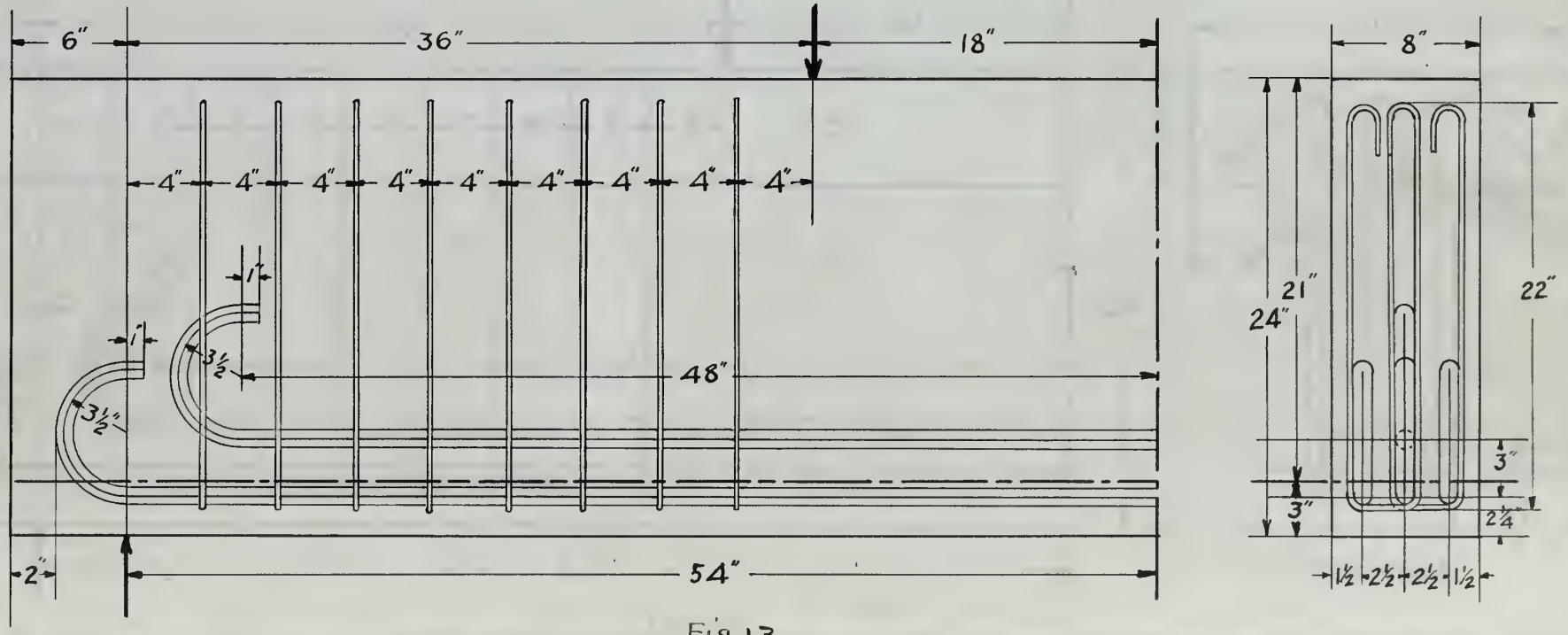
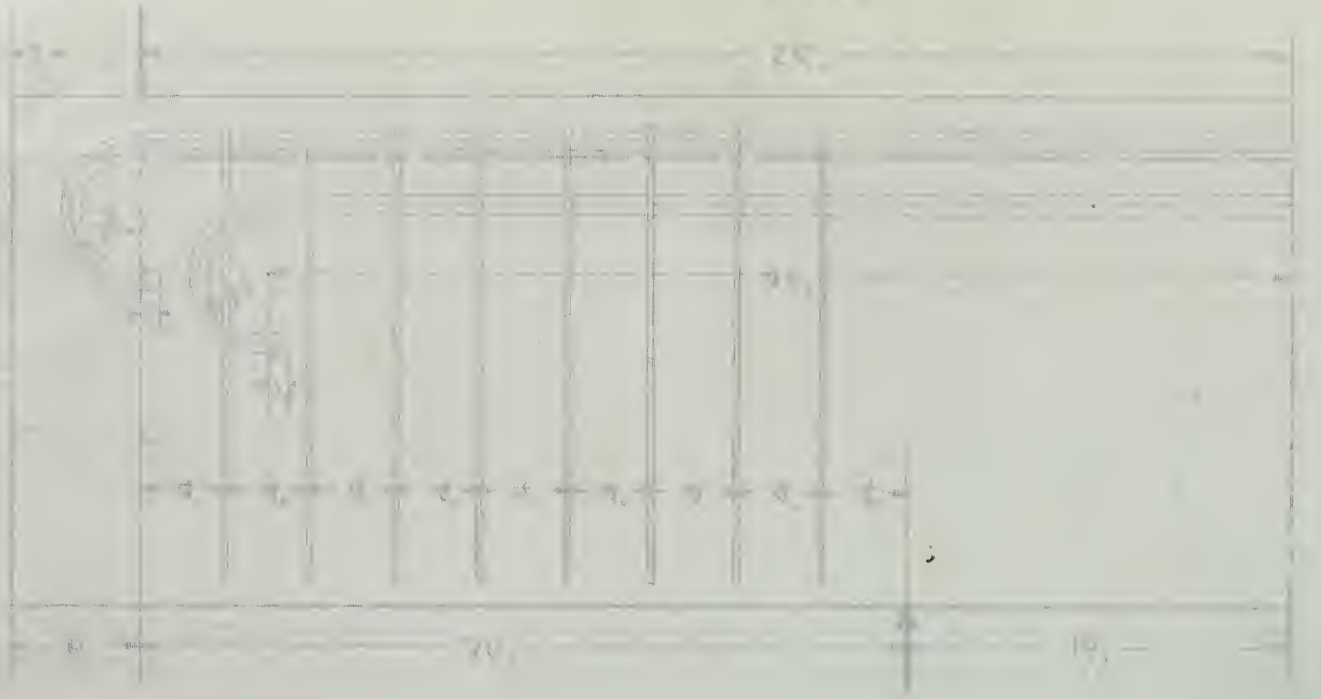
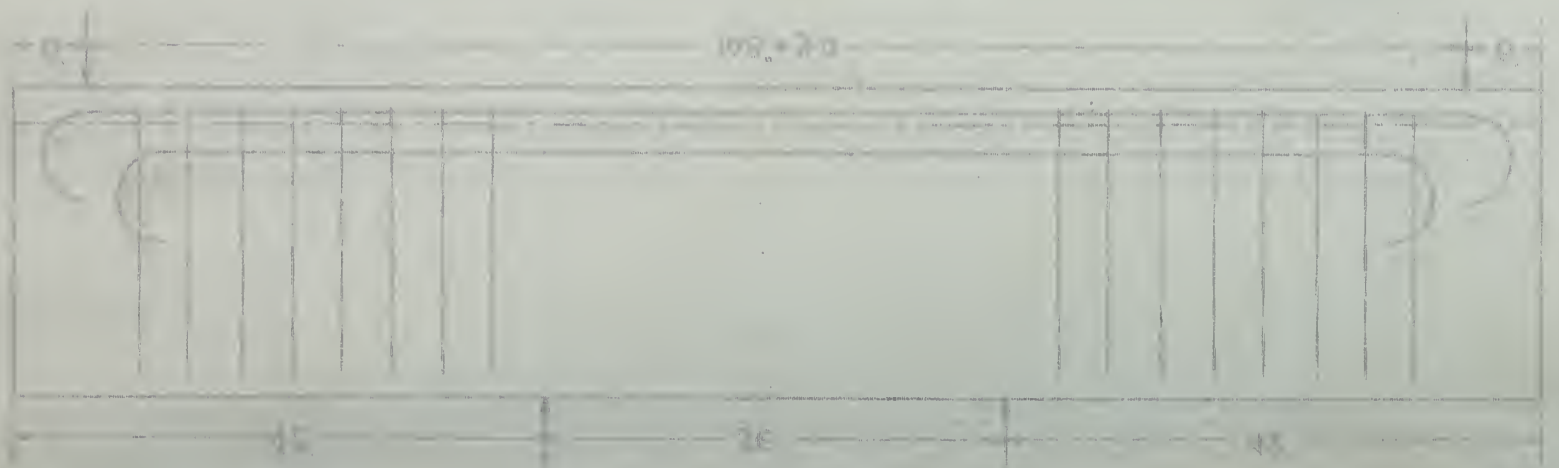


Fig 13





Beam No. 22,6

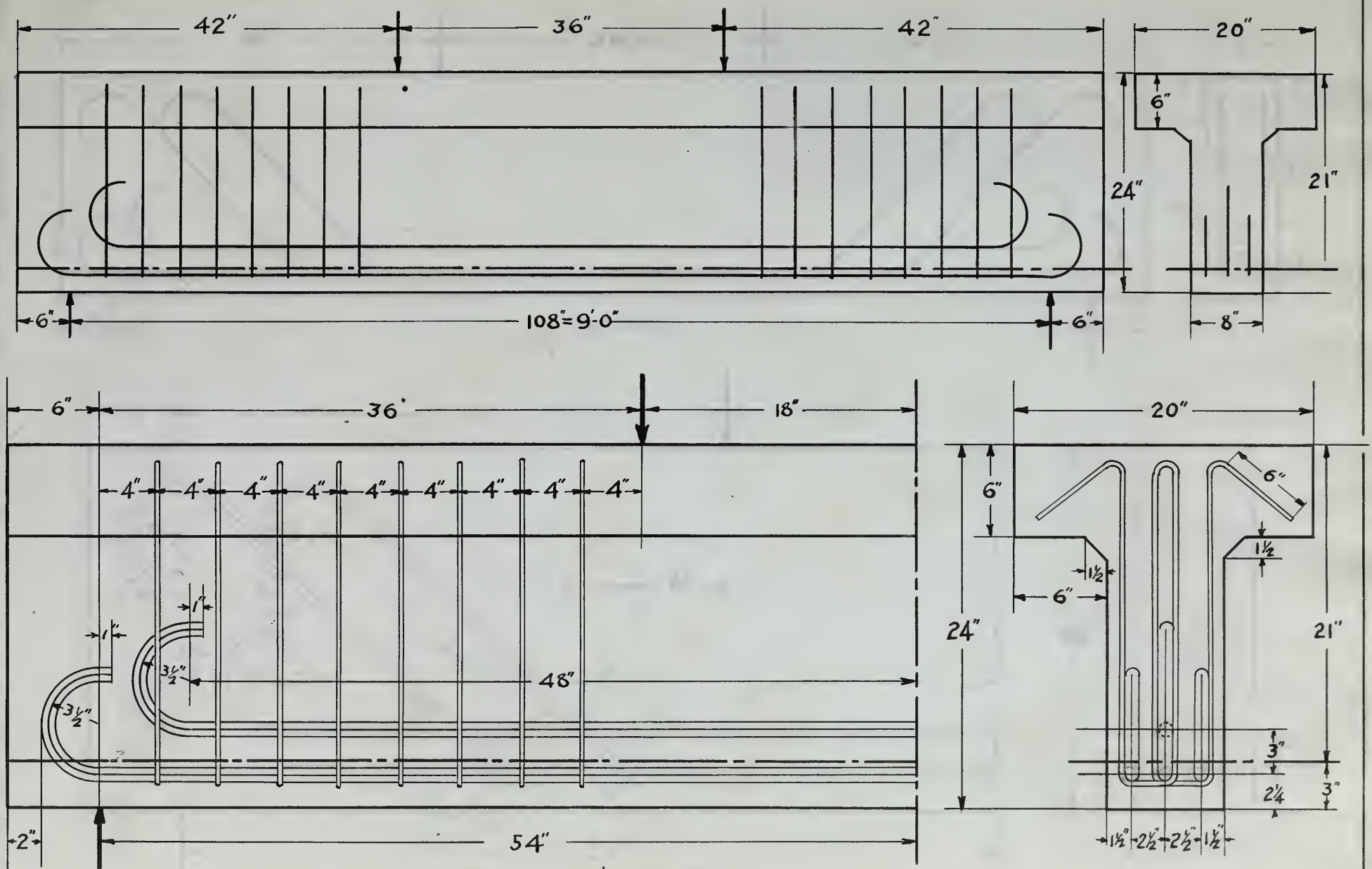
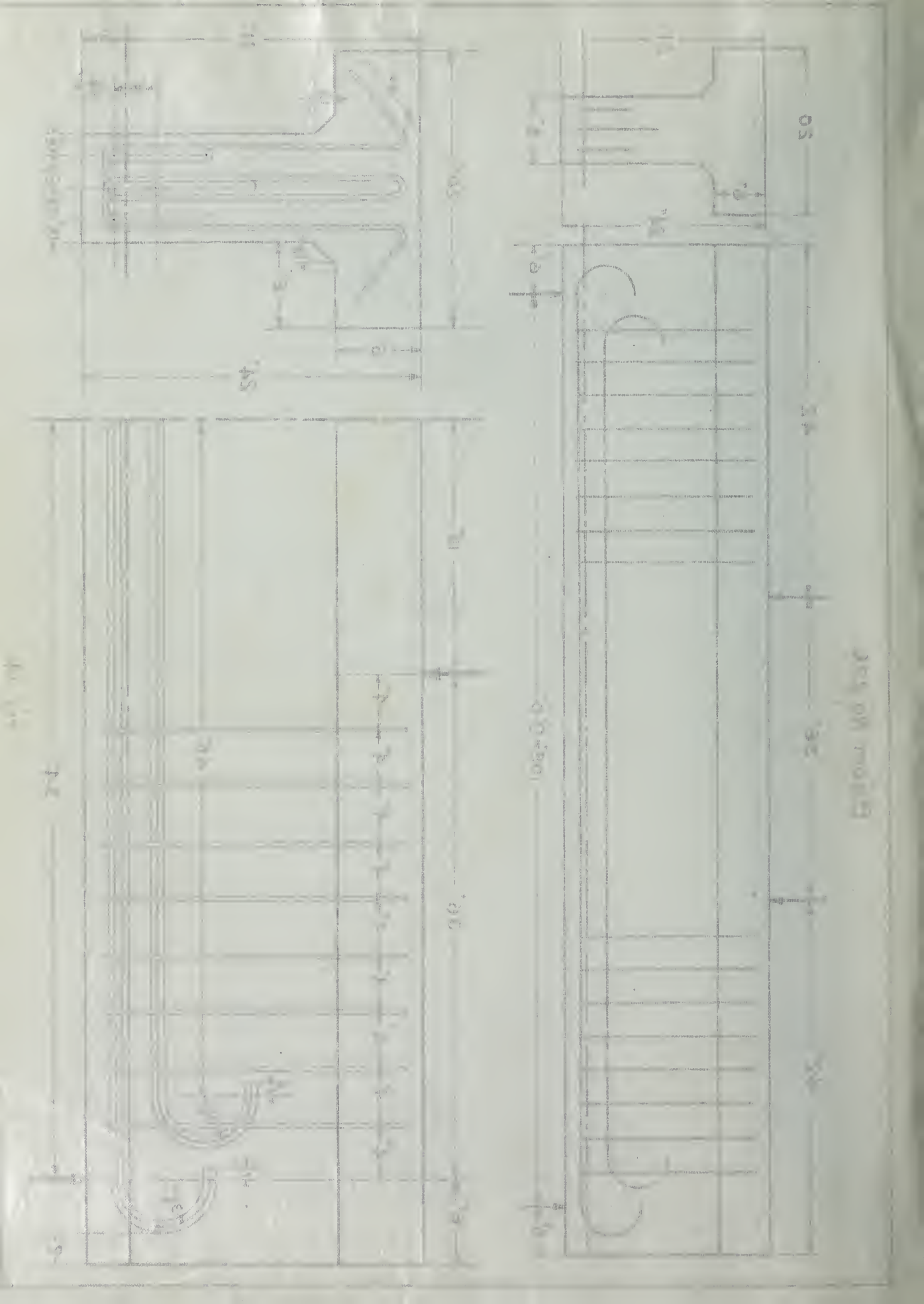


Fig. 14



Beam No. 22.9

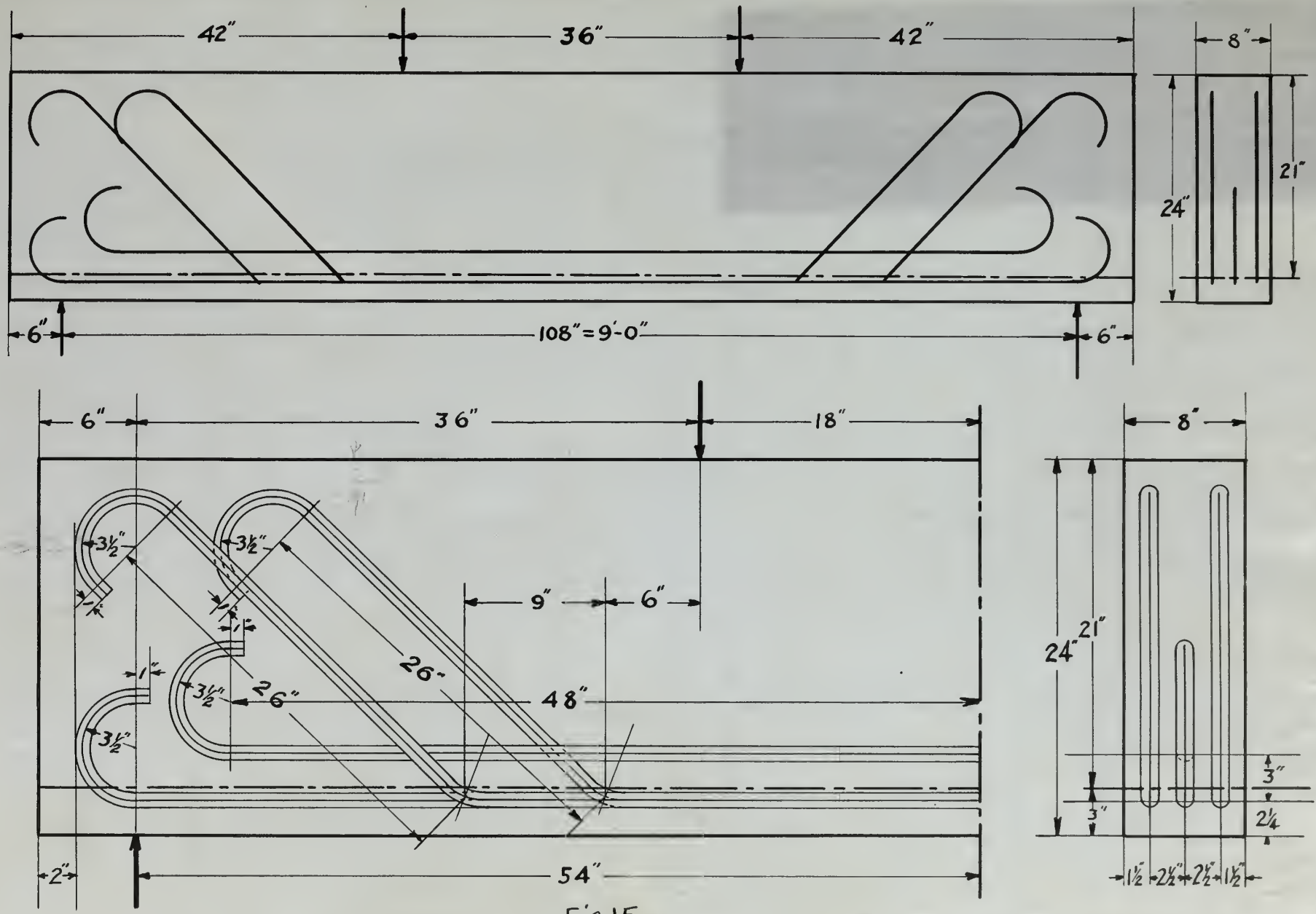
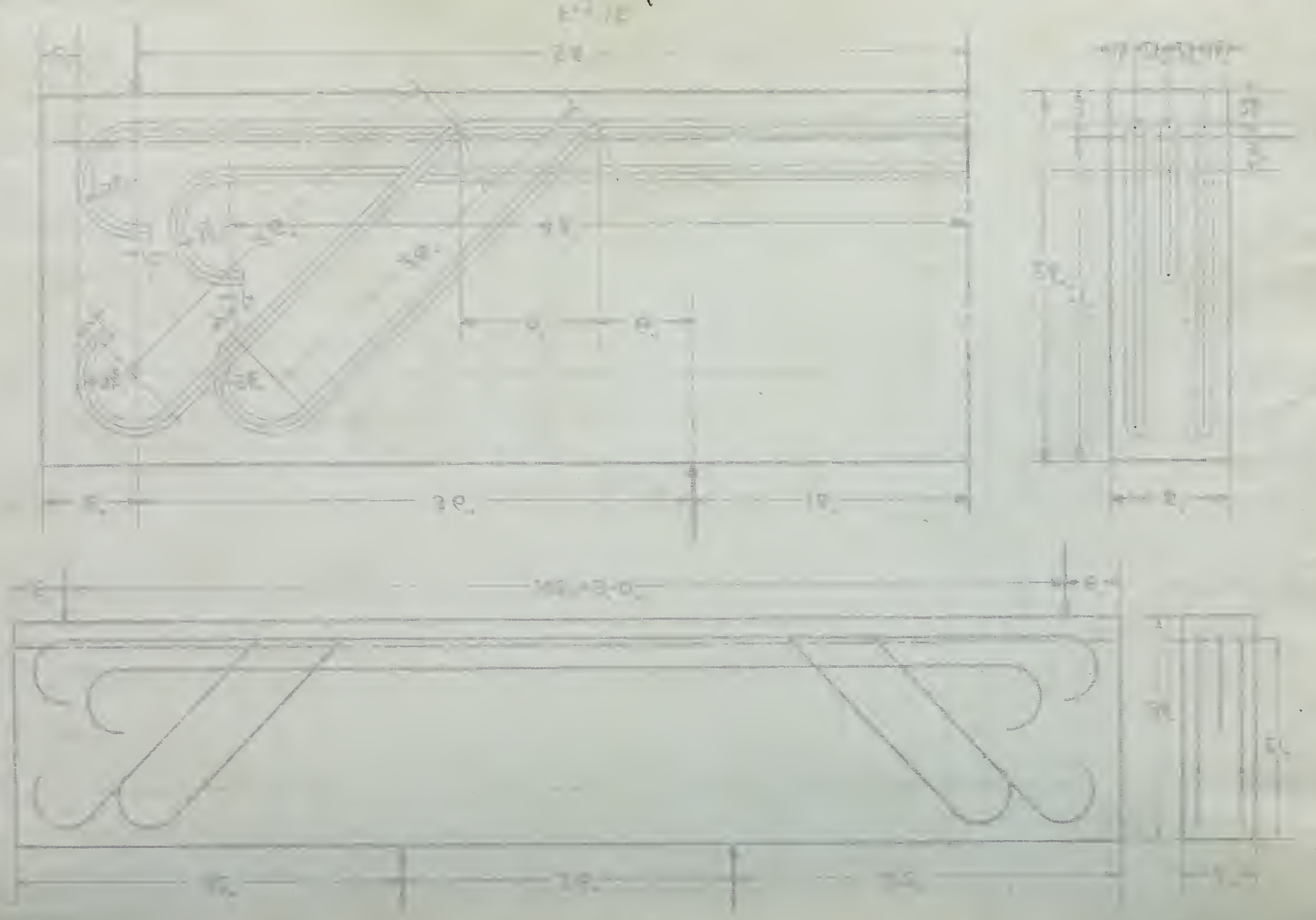
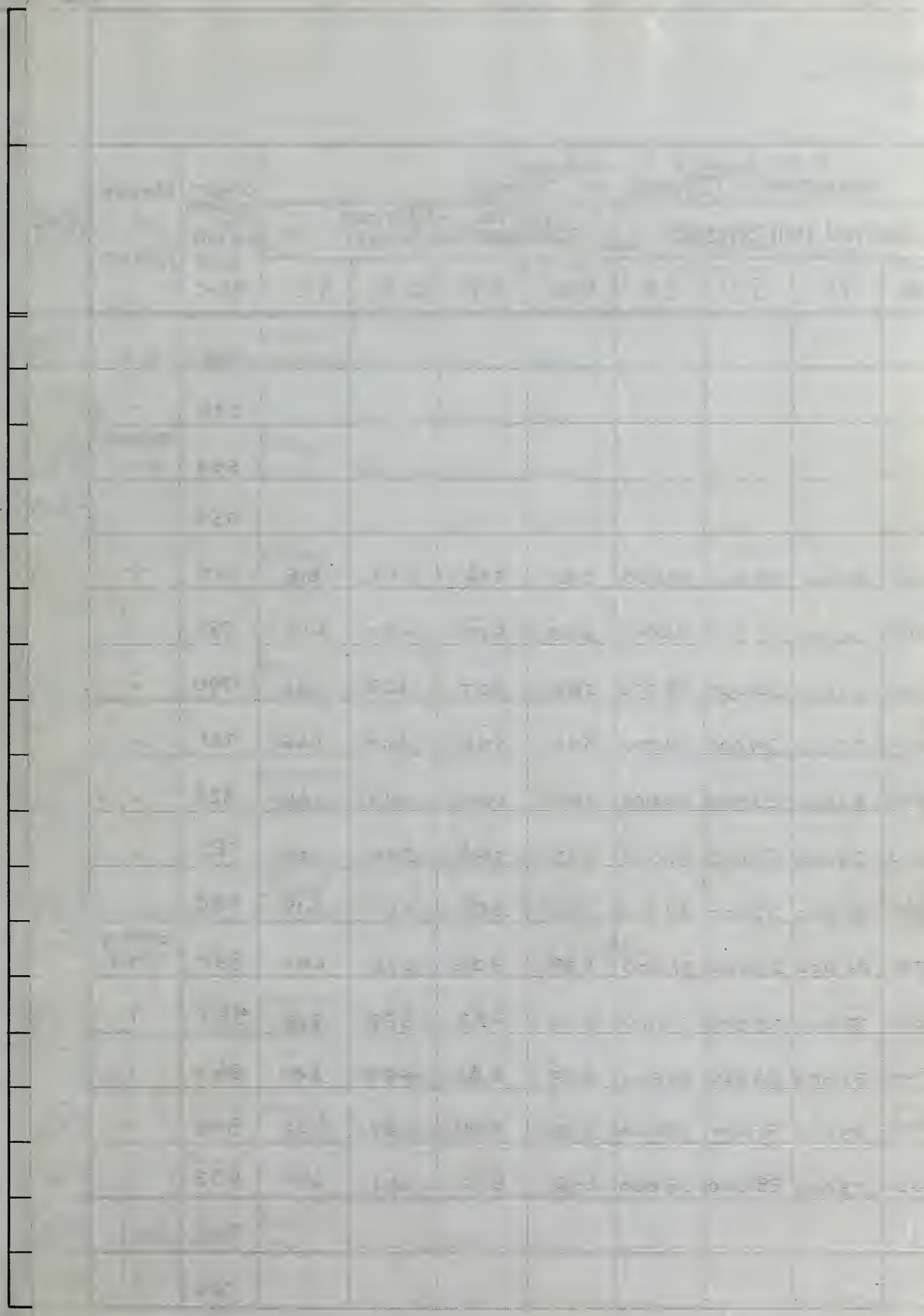


Fig 15





Summary

Beam No.	Section	Variety.	Age	Maximum Load pounds	6x12 in. Cylinder Tests Unit Load pounds	Stirrups. Spacing in	Size	Sectional Area in²	Surface Area in²	At the Load of 200 000 lbs Maximum Stresses in Stirrups. Calculated Unit Stress	Observed Unit Stresses N.W.	N.E.	S.W.	S.E.	Observed Unit Stresses / Calculated Unit Stresses N.W.	N.E.	S.W.	S.E.	Shearing Stress at the Ulti. Load lb/in²	Manner of Failure
22.1-1	Rectangular	Without hooks & stirrups	61	154 100	4076														552	D.T.
-2	"	"	61	150 000	3696														538	"
22,2-1	"	Without stirrups	60	167 500	4522														599	Breaking in shear
-2	"	"	59	128000	4337														459	"
22.3-1	"	Stirrups 4" Spacing	60	214 550	4124	4	3/8	.1104	1.178	52 000	30 000	31 000	34 000	32000	.577	.596	.654	.615	769	T
-2	"	"	61	218000	3689	"	"	"	"	"	35 500	30 000	25 500	32000	.690	.577	.487	.615	.781	"
22.4-1	"	Stirrups 7" Spacing	62	220500	4106	7"	1/2	.1967	1.571	51400	30 000	26000	32000	28 000	.586	.507	623	.546	790	"
-2	"	"	61	218000	3790	"	"	"	"	"	30 000	27000	34000	29000	.586	.526	603	.566	781	"
22.5-1	"	Stirrups 11" Spacing	62	229300	3788	"	5/8	.3067	1.963	51400	30 000	27000	23000	24000	.582	.524	.447	.466	822	"
-2	"	"	61	214500	4041	"	"	"	"	"	31000	28000	28000	27000	.603	.544	.544	.524	769	"
22.6-1	T-shaped	Stirrups 4" Spacing	62	262340	4037	4	3/8	.1104	1.178	48 500	32500	30 000	27000	30000	.671	.619	.557	.619	940	
-2	"	"	60	248300	4331	"	"	"	"	"	31000	26 000	28000	31000	.639	.536	.578	.641	890	Bearing Failure
22,7-1	"	Stirrups 7" Spacing	62	261 300	3799	7	1/2	.1967	1.571	47 900	35000	33 000	25000	39 000	.734	.692	.528	.818	937	T
-2	"	"	60	268600	4346	"	"	"	"	"	31000	32 500	23000	30500	.650	.681	483	.640	963	"
22,8-1	"	Stirrups 11" Spacing	62	264200	4058	11	5/8	3067	1.963	47 800	24500	29000	31 000	29000	.512	.649	.397	.605	948	"
-2	"	"	60	260000	4152	"	"	"	"		29000	23 000	29 500	33000	.606	.616	481	690	932	"
22. 9-1	Rectangular	Bent up Bars	62	225400	3931														808	"
-2	"	"	60	213 700	4203														766	"

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